

# CONCRETE

## AND CONSTRUCTIONAL ENGINEERING

MAY, 1950.



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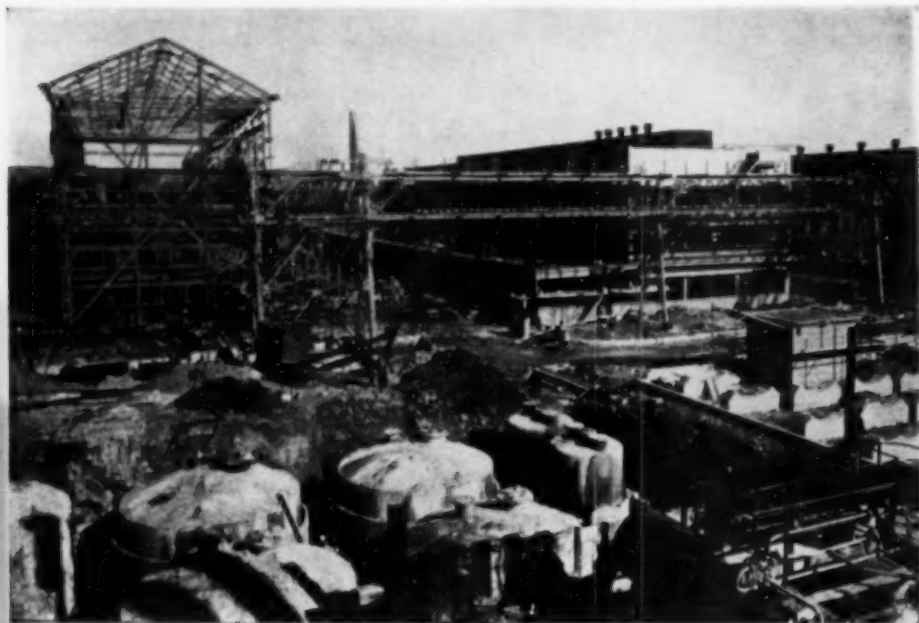
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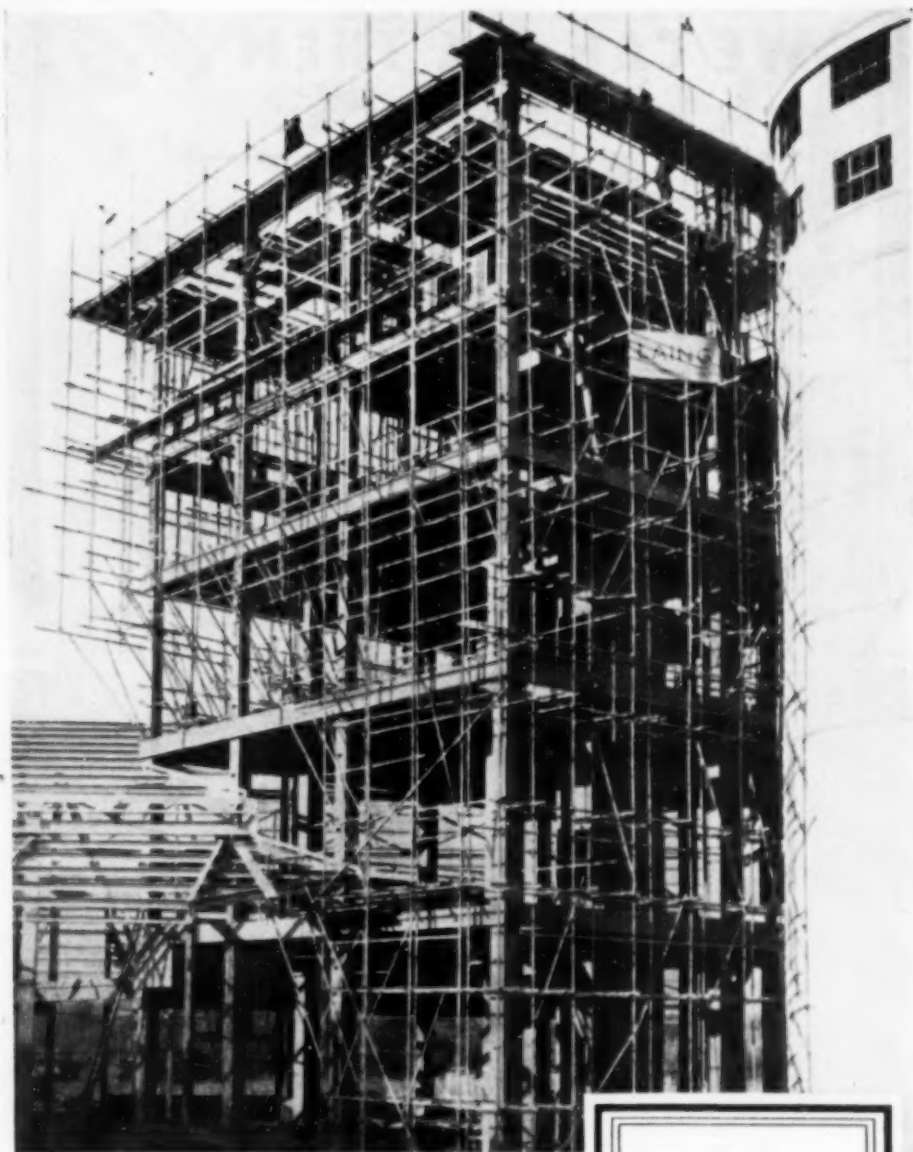
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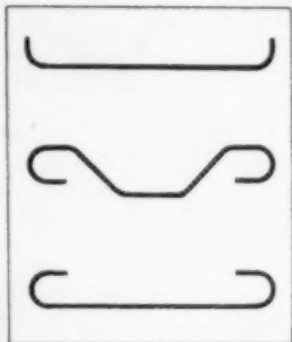
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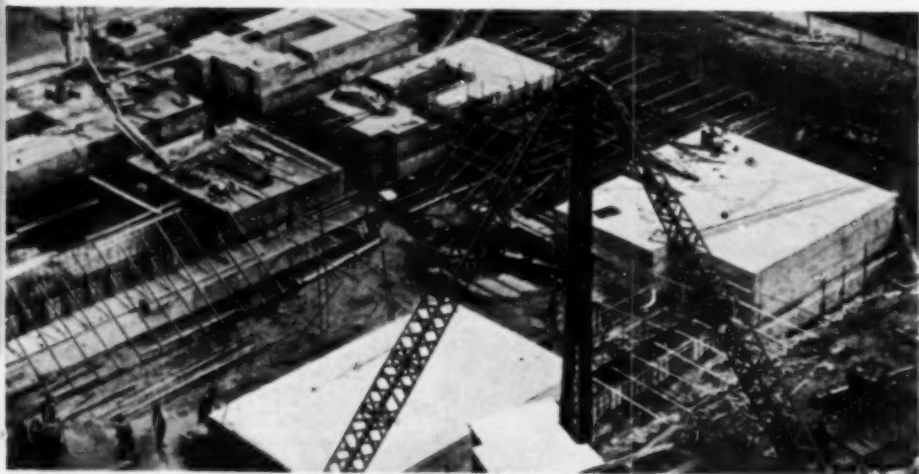
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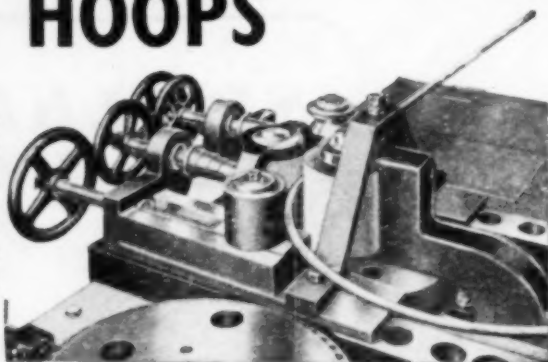
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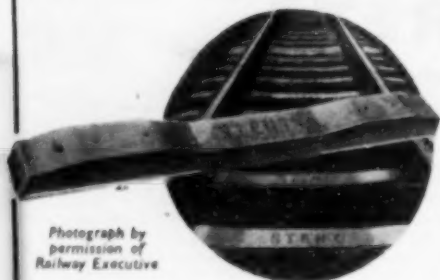
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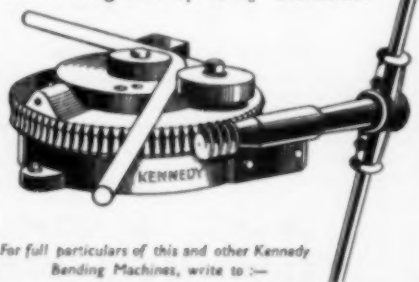
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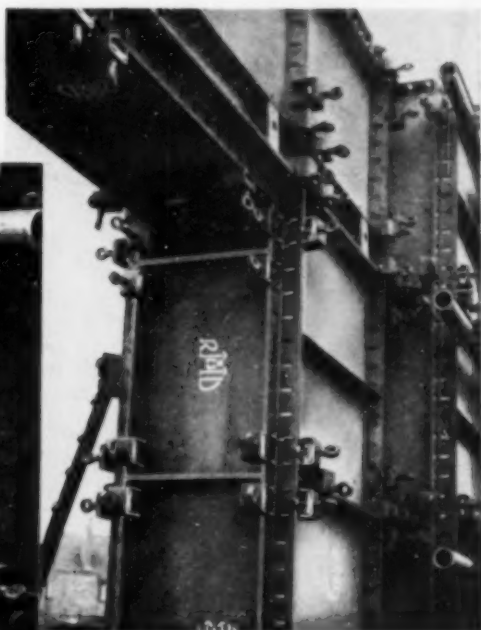
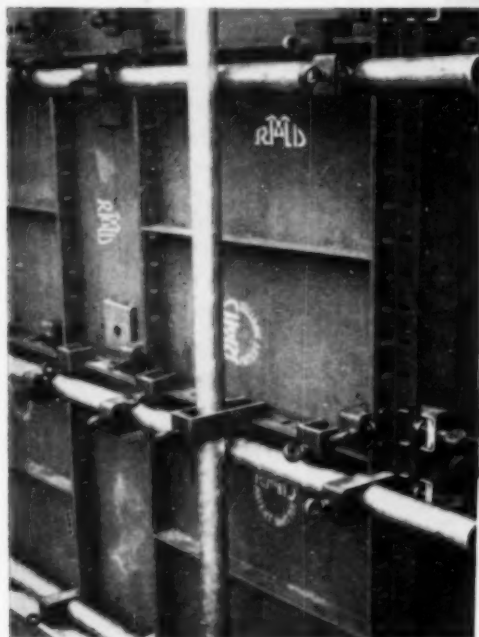
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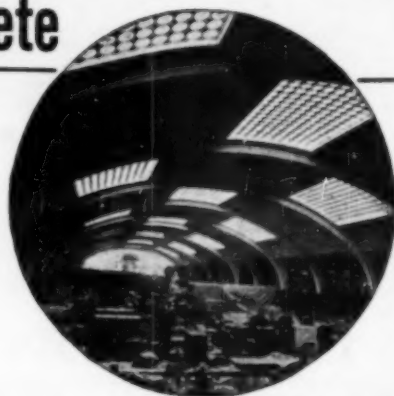
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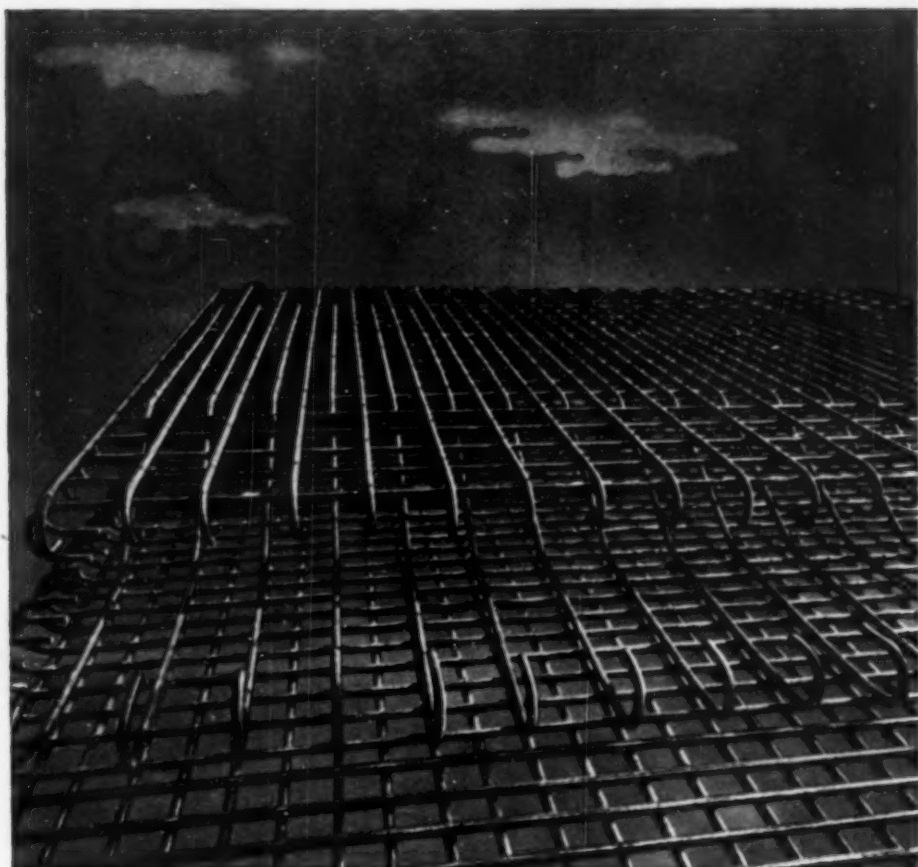


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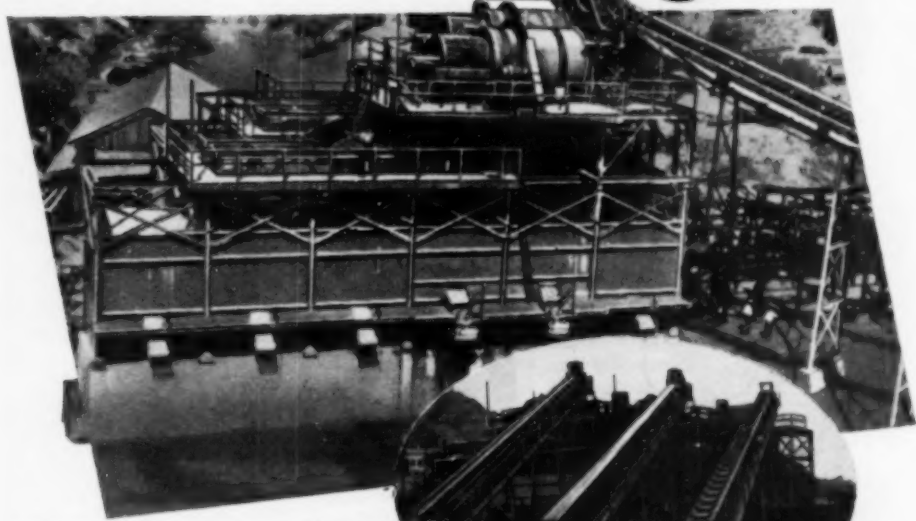
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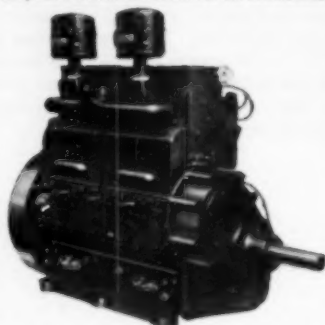
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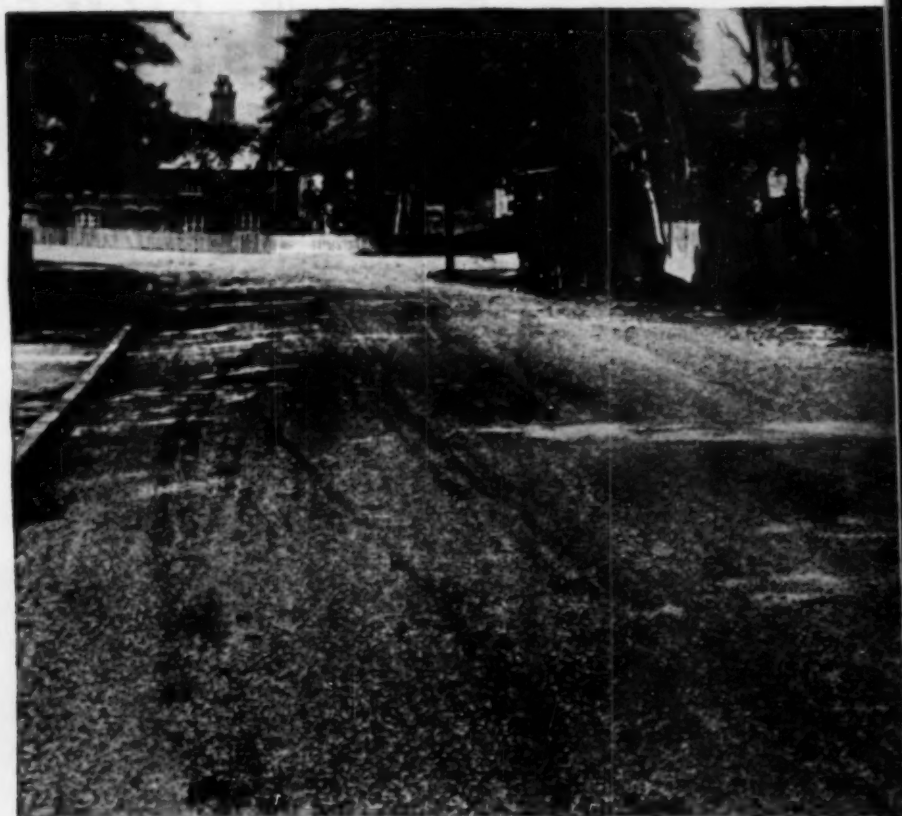
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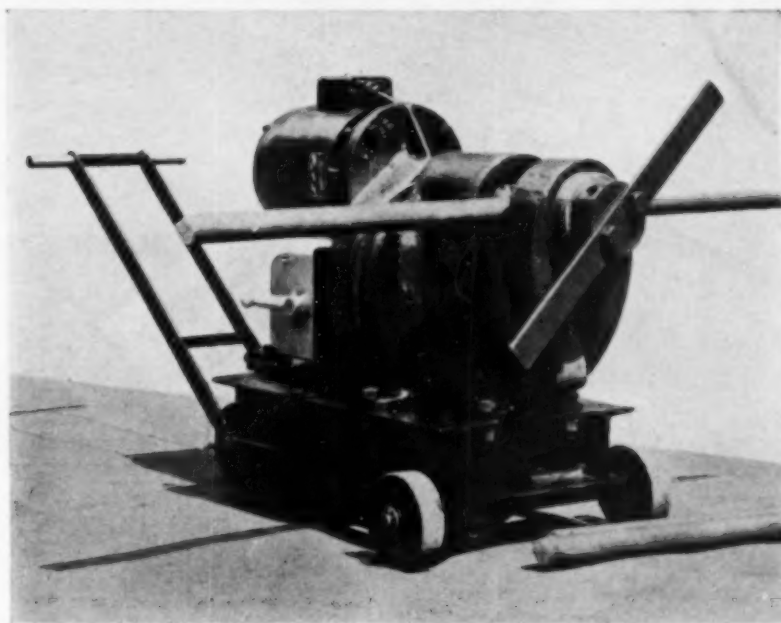
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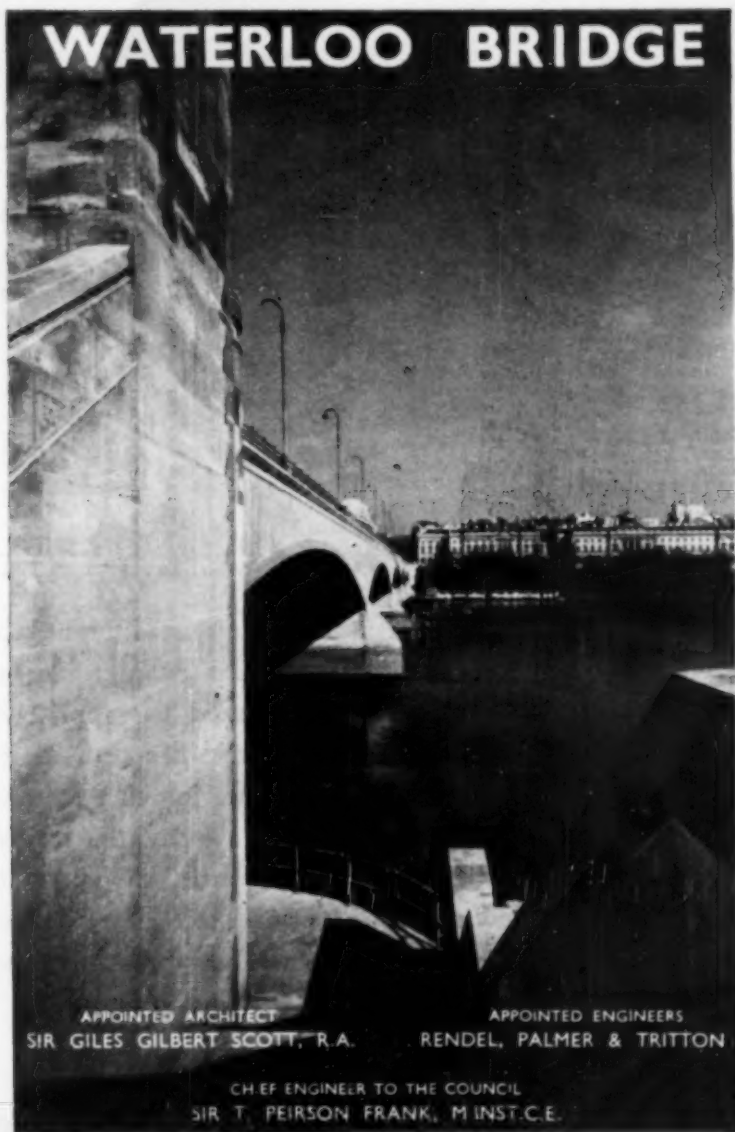
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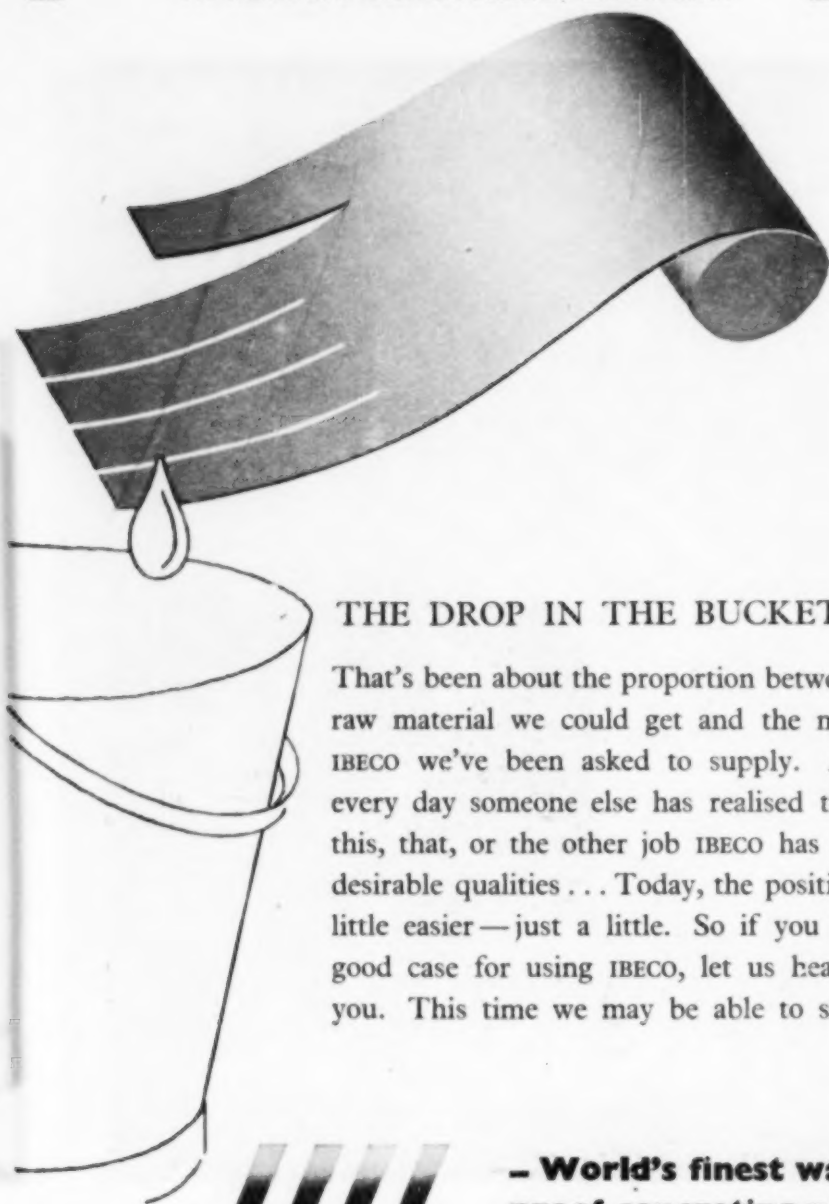
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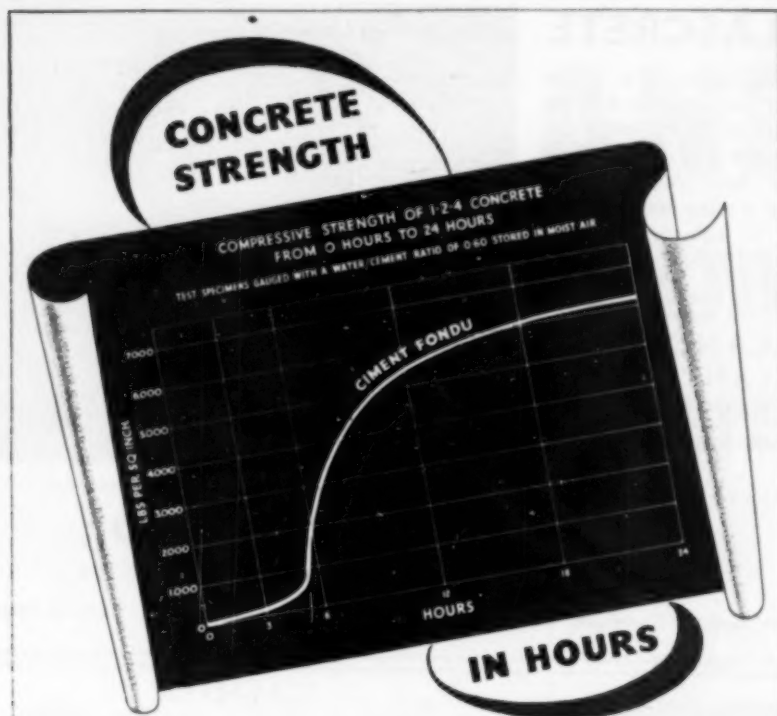


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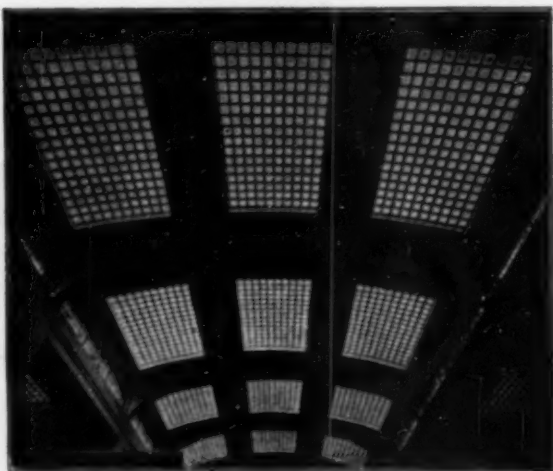


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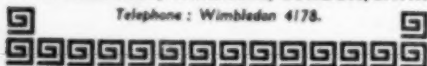
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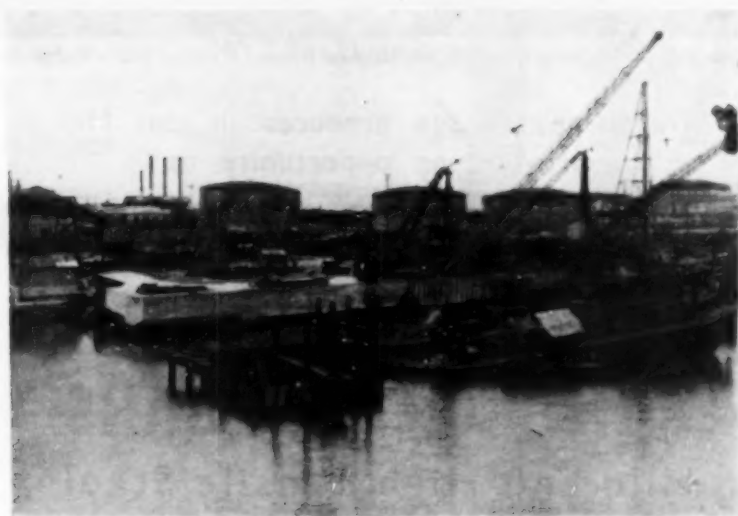
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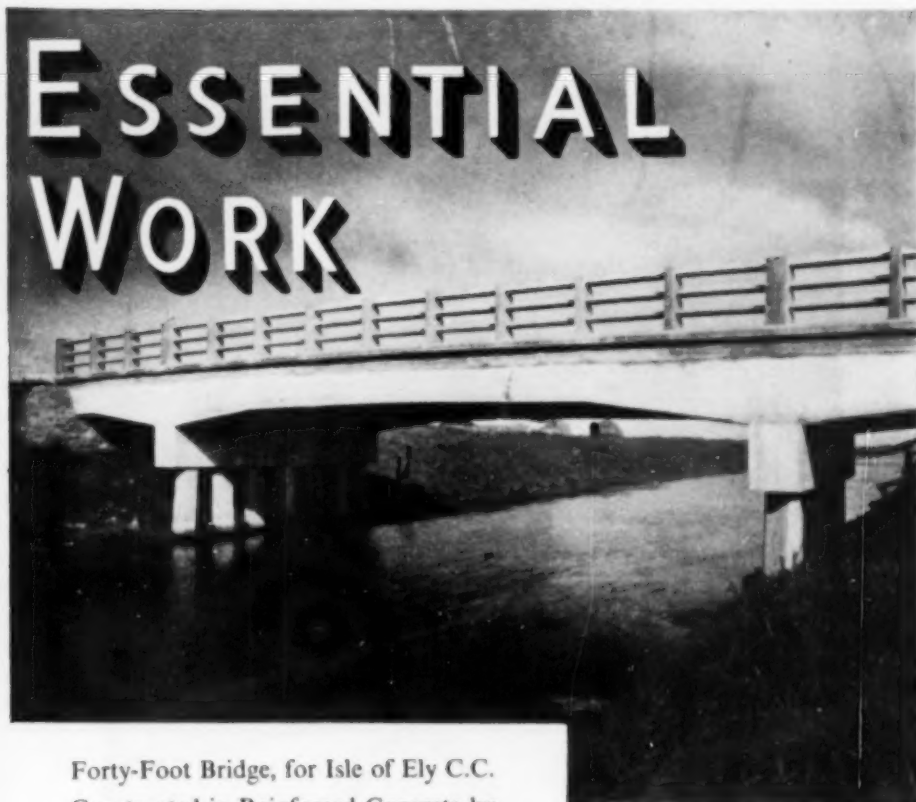
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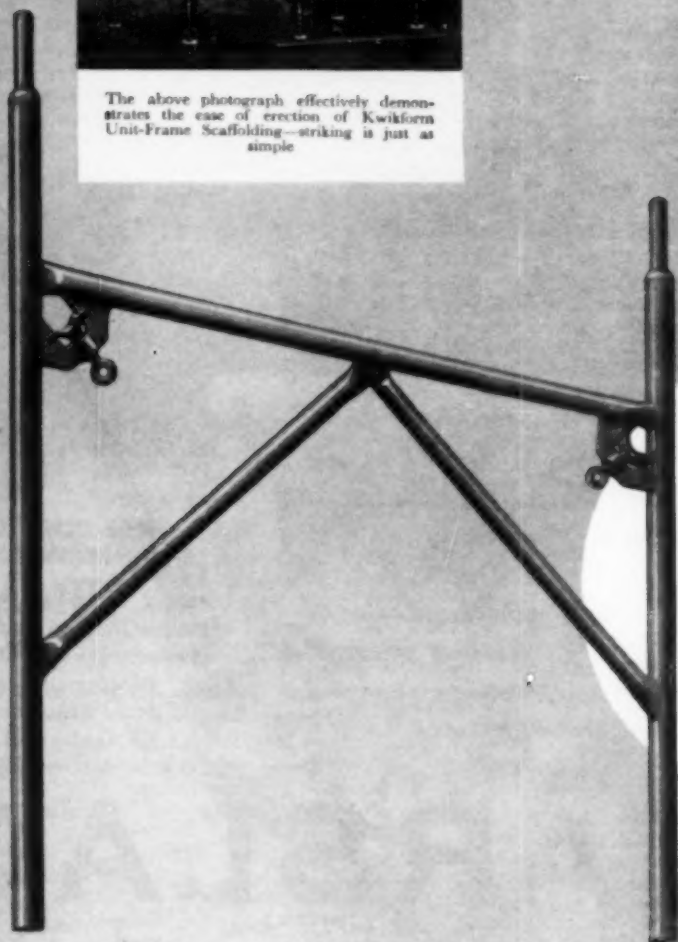
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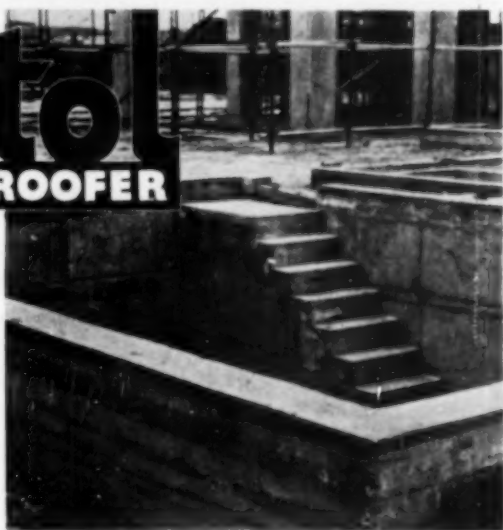
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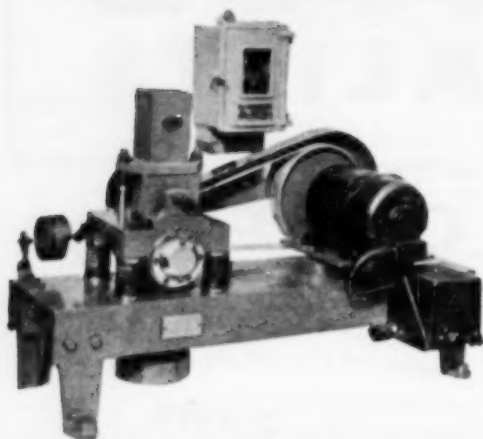
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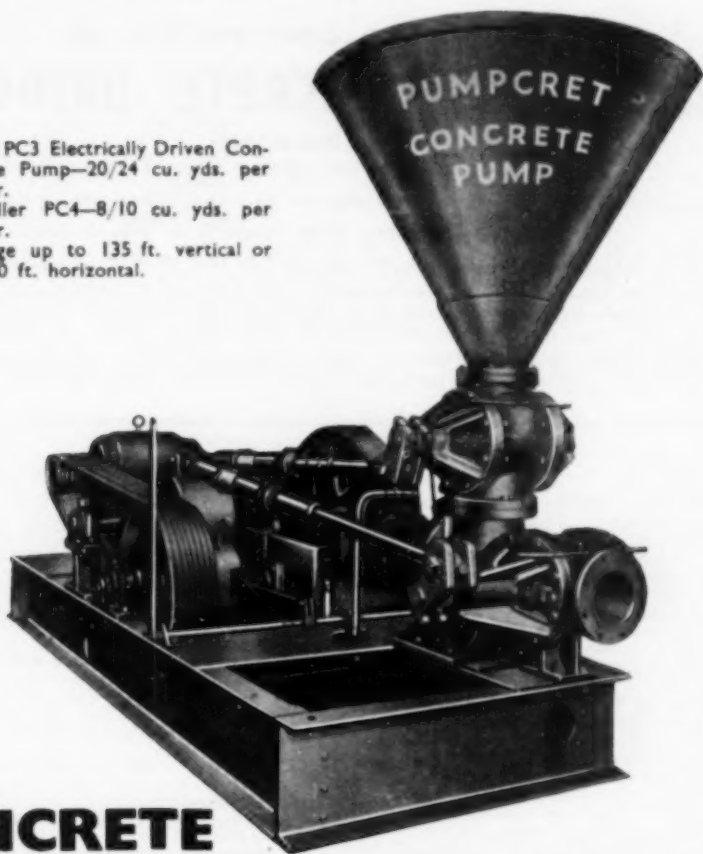
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# CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume XLV. No. 5.

LONDON, MAY, 1950

## EDITORIAL NOTES

### Learning and Skill.

THE Editorial Note in our January number, in which we urged that a higher standard of general education than is at present common was desirable before a young man started to concentrate on special training for his profession, brought a number of replies. Without exception these deprecated our suggestion that a course in the liberal arts should precede technical training, on the grounds of the extra expense and the longer time that would be spent at college. Since then the subject has been debated at some length in Parliament and elsewhere, and our view has been supported by many eminent men. Perhaps Lord Chorley, in the House of Lords, pointed out most clearly one of the main problems when he said: "There is a very important distinction between the technologist and the technician, and I rather fear that there is a tendency to confuse the two. A technologist is a man who engages in the scientific study of the practice of industrial arts, and in the practical application of scientific principles and discoveries to the arts and to industry. In other words, he is a man who is concerned with the fundamental scientific principles on which technology rests. He is the commissioned officer of the industrial army, and it is from among these commissioned officers that more and more of the field officers of industry and its leadership generally are being drawn. The technician, on the other hand, is a person who carries out in a responsible manner approved techniques which are either common knowledge or specially prescribed by the management of the business. In other words, the technician is the non-commissioned officer of the industrial army." Or, as we expressed it, on the one hand there are scientists and engineers, and on the other hand laboratory assistants, designer-draughtsmen, computers of stresses, and others doing routine work in laboratories or in engineers' offices or practising as consulting engineers in the class of everyday work that needs no special degree of vision or special sense of engineering. With a few fortunate exceptions, the education of all would-be scientists and engineers is the same, that is a general education up to the stage of matriculation in their early 'teens followed by intensive study of special subjects. The result may be a degree in science and sufficient knowledge to enable a man to get employment, but it is not education in the true sense of the word. Most of our correspondents expressed the view that this is all that is necessary, and so it is if the aim is to produce only "non-commissioned officers," men who are technicians only, of whom "The Times" recently wrote, "It is a constant complaint from industry that many of them are unable to draft clearly a simple report of work for which

they are responsible." But it will not produce—except by chance—men with a true scientific outlook.

The view that we have often expressed was recently emphasised by Dr. Percy Dunsheath, C.B.E., D.Sc., M.A., a director of an important industrial company. "It is essential," he wrote, "to get clear the difference between technical training and the education of technologists. The requirements for the education of the technologist and the requirements of industry are exactly the same—education in the fundamentals of science to a high standard, for technology is not, as is commonly supposed, concerned merely with the 'know-how' of industry but with the application of scientific principles and the scientific method to solve current problems and to build up new branches of industry. But equally important is the question of general education. Industry is looking more and more for technologists in positions of leadership and of high administrative responsibility. In order that they may acquire the vision and the wide grasp of affairs which such positions call for they need a background of more general culture, such as economics, languages, history, and philosophy. This has long been recognised in the foreign institutes, and of recent years by some of our own technological faculties."

Dr. Dunsheath suggests that this kind of education is best undertaken by the universities, while the technical training is best carried out by the technical colleges. A report issued by the Advisory Committee on Scientific Policy (see this journal for October 1948) stated that "scientists" were being produced almost at the rate of 5000 a year, and the only justification for the claim was that nearly 5000 people were awarded a degree in science in the previous year. Some of these men may be potential scientists, but the number is very few because they have not the wider and more liberal education which is necessary in a scientist; they will be skilled in the routine work of one branch of science and be useful members of society, just as are men who are trained and become skilled workers in any other walk of life.

The narrow outlook that results from technical training without a liberal education is frequently to be noted in the engineering profession. We see, for example, in the Journal of the Engineers' Guild a claim that a man with a degree in engineering or who is a member of one of the premier engineering societies is "a properly and adequately qualified engineer and scientist." How often is this true? How many such men are competent to initiate and carry through a major work of engineering? By far the majority of them will for the rest of their lives be working as routine designers, either on their own account or in the employ of others, because their knowledge and ability do not fit them for more responsible work. In the present system there is insufficient provision for giving extra help to men whose work shows that they have more than the average amount of natural ability. If he has not an exceptional degree of natural ability no amount of education or training will make a man a brilliant scientist or engineer. Exceptional natural ability will, but rarely, produce exceptional scientists at an early age; for example, Marconi had but little school training when he transmitted wireless messages in his early twenties. In most cases a liberal education will be of untold help to men with natural ability. At present this is nearly always limited by the means of the parents and the need to start earning at the earliest possible age.

## Influence Lines for Continuous Structures.

By L. A. BEAUFOY, Ph.D., M.Sc., A.M.Inst.C.E., M.I.Mech.E., M.I.Struct.E.

INFLUENCE lines depend only on the elastic properties and shape of a structure and are independent of the load. The deflection diagram resulting from unit displacement imposed at any point in the direction of a stress component, such as a bending moment or shearing force, is identical with the influence line for the stress component. This principle is the basis of some experimental determinations of influence lines by models. In the "Quarterly Journal of Mechanics and Applied Mathematics," September, 1949, Dr. A. F. S. Diwan and the writer describe a procedure for evaluating directly the elastic properties of a structure in terms of the displacement of the joints due to unit relative displacement imposed

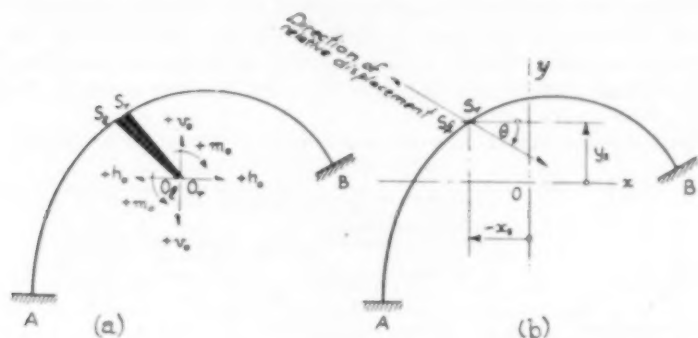


Fig. 1.

at any point, and in the following it is shown how the procedure may be adapted to the analytical determination of influence lines for continuous structures without the use of simultaneous equations or successive approximations.

Consider the continuous elastic member AB in Fig. 1 (a) fixed at both ends, and suppose that the influence line for an internal redundant at section  $S_1S_2$  is required. It is imagined that the member is cut at this section, and that rigid arms  $S_1O_1$  and  $S_2O_2$  connect the cut ends to the elastic centre O. The problem is then to determine the values of the moment  $m_0$  and the horizontal and vertical forces  $h_0$  and  $v_0$  which should be imposed at the ends  $O_1$  and  $O_2$  to produce unit relative displacement at  $S_1$  and  $S_2$ . These values follow from the equations, the derivation of which is given in the paper referred to,

$$\phi_0 = m_0 \bar{S}; \Delta_0 = h_0 I_x - v_0 I_{xy}; \lambda_0 = v_0 I_y - h_0 I_{xy} \quad (1)$$

where  $\phi_0$ ,  $\Delta_0$ , and  $\lambda_0$  are the rotational, horizontal, and vertical displacements at the elastic centre,  $\bar{S}$ , the elastic area of the member,  $I_x$  and  $I_y$ , the centroidal moments of inertia of the member about the axes of  $x$  and  $y$  respectively, and  $I_{xy}$ , the centroidal product of the moments of inertia about the axes of  $x$  and  $y$ . The signs for the forces on the left-hand and right-hand rigid arms at the elastic centre are shown in Fig. 1 (a). Positive relative displacement is produced

by positive bending moment or force. From equations (1), the moments and forces at the elastic centre are

$$m_0 = \frac{\phi_0}{S}; \quad h_0 = \frac{A_0 I_y + \lambda_0 I_{xy}}{I_x I_y - I_{xy}^2}; \quad v_0 = \frac{A_0 I_{xy} + \lambda_0 I_x}{I_x I_y - I_{xy}^2} \quad (2)$$

If  $A_0' = A_0 + \lambda_0 \frac{I_{xy}}{I_y}$ ,  $\lambda_0' = \lambda_0 + A_0 \frac{I_{xy}}{I_x}$ ,  $I_x' = I_x - \frac{I_{xy}^2}{I_y}$ , and  $I_y' = I_y - \frac{I_{xy}^2}{I_x}$ , equations (2) become

$$m_0 = \frac{\phi_0}{S}; \quad h_0 = \frac{A_0'}{I_x'}; \quad v_0 = \frac{\lambda_0'}{I_y'} \quad (3)$$

Hence for the general case of unit relative displacement at section  $S_1 S_2$  in a given direction [Fig. (1b)], it can be shown that the required centroidal moment and forces are as given in column (2) in Table No. I. Values for special cases such as unit rotation, unit horizontal translation, and unit vertical translation, which are given in columns (3), (4), and (5) respectively, are derived from the values for the general case, and are the values required to distort the member in such a way as to obtain directly the influence lines for bending moment, horizontal

TABLE I—CENTROIDAL MOMENT AND FORCES IN TERMS OF RELATIVE DISPLACEMENT.

		DIRECTION OF IMPOSED RELATIVE DISPLACEMENT			
		GENERAL CASE OF UNIT RELATIVE DISPLACEMENT AT $\theta$ DEG. TO HORIZONTAL	UNIT ROTATION	UNIT HORIZONTAL TRANSLATION $\theta = 0$ DEG.	UNIT VERTICAL TRANSLATION $\theta = 270$ DEG.
(1)		(2)	(3)	(4)	(5)
RELATIVE DISPLACEMENT IMPOSED AT SECTION $S_1 S_2$	$\Delta_0$	—	1.0	—	—
	$\Delta_0$	$\cos \theta$	—	1.0	—
	$\lambda_0$	$-\sin \theta$	—	—	1.0
CORRESPONDING RELATIVE DISPLACEMENT AT ELASTIC CENTRE	$\beta_0$	—	1.0	—	—
	$\Delta_0$	$\cos \theta$	$-\gamma_0$	1.0	—
	$\lambda_0$	$-\sin \theta$	$x_0$	—	1.0
CORRECTED VALUES	$\beta_0'$	—	1.0	—	—
	$\Delta_0'$	$\cos \theta - \sin \theta \left( \frac{I_{xy}}{I_y} \right)$	$-\gamma_0 + x_0 \left( \frac{I_{xy}}{I_y} \right)$	1.0	$\frac{I_{xy}}{I_y}$
	$\lambda_0'$	$-\left[ \sin \theta - \cos \theta \left( \frac{I_{xy}}{I_x} \right) \right]$	$x_0 - \gamma_0 \left( \frac{I_{xy}}{I_x} \right)$	$\frac{I_{xy}}{I_x}$	1.0
CENTROIDAL MOMENT AND FORCES	$m_0$	—	$\frac{1.0}{S}$	—	—
	$h_0$	$\frac{(\cos \theta)'}{I_x'}$	$\frac{-\gamma_0'}{I_x'}$	$\frac{1.0}{I_x'}$	$\frac{I_{xy}}{I_y I_x'}$
	$v_0$	$\frac{-(\sin \theta)'}{I_y'}$	$\frac{x_0'}{I_y'}$	$\frac{I_{xy}}{I_x' I_y'}$	$\frac{1.0}{I_y'}$
$x_0' = x_0 - \gamma_0 \left( \frac{I_{xy}}{I_x} \right)$		AXIAL THRUST OR	BENDING	HORIZONTAL	VERTICAL
$\gamma_0' = \gamma_0 - x_0 \left( \frac{I_{xy}}{I_y} \right)$		NORMAL SHEARING FORCE (APPROPRIATE $\theta$ )	MOMENT	FORCE	FORCE
$(\cos \theta)' = \cos \theta - \sin \theta \left( \frac{I_{xy}}{I_y} \right)$		CORRESPONDING INFLUENCE LINE			
$(\sin \theta)' = \sin \theta - \cos \theta \left( \frac{I_{xy}}{I_x} \right)$					

force, and vertical force respectively at the section  $S_1S_2$ . The influence lines for axial thrust or shearing force at the section are obtained by substituting the appropriate value of  $\theta$  in the general expressions given in column (2). When the values of  $m_0$ ,  $h_0$ , and  $v_0$  are known, the bending moment  $m$  at any other section  $(x, y)$  in the member is

$$m_0 - h_0 y + v_0 x \quad (4)$$

in which positive bending moment is that producing tension on the inside of the member AB. Hence a diagram of the bending moment  $m$  is obtained corresponding to the imposed deformation. Considering the  $\frac{m}{EI}$ -diagram as the load, the deflection diagram for the member is obtained by the moment-area

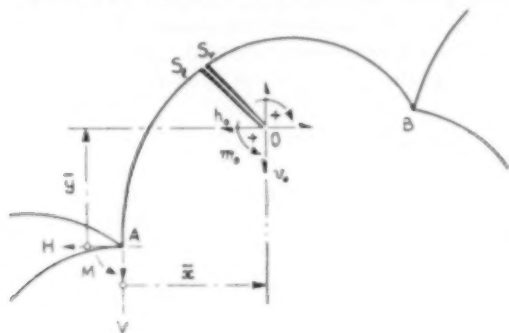


Fig. 2.

method and, according to the Müller-Breslau principle, is the influence line for the stress component corresponding to the imposed deformation. In fact the deflection diagram may be said to result from the indeterminate moments and forces caused by the imposed deformation, assuming the ends of the member to be fixed.

Now let AB be a member forming part of a continuous structure (Fig. 2). In this case, joints A and B, instead of being fixed, are free to rotate and translate under the effects of an imposed relative deformation at  $S_1S_2$  as an expression of the elastic characteristics of the whole structure. The centroidal moment and forces created by the imposed deformation cause an unbalanced action on joints A and B which can be obtained by statics. Thus at A the unbalanced moment  $M$  and the horizontal and vertical forces  $H$  and  $V$  respectively are given by

$$M = -m_0 - h_0 \bar{y} + v_0 \bar{x}; \quad H = -h_0; \quad V = -v_0 \quad (5)$$

where  $\bar{x}$  and  $\bar{y}$  are the horizontal and vertical co-ordinates respectively of the elastic centre O relative to joint A. The unbalanced moment and forces cause at joint A balancing movements  $\phi$ ,  $\Delta$ ,  $\lambda$  (rotationally, horizontally, and vertically respectively) which can be written

$$\phi = M\phi_m + H\phi_h + V\phi_v; \quad \Delta = M\Delta_m + H\Delta_h + V\Delta_v; \quad \lambda = M\lambda_m + H\lambda_h + V\lambda_v \quad (6)$$

in which, for example,  $\phi_m$  represents the rotational displacement due to  $M = 1$ ,  $\phi_h$  that due to  $H = 1$ , and  $\phi_v$  that due to  $V = 1$ . The methods of determining these displacement coefficients are given in the paper referred to before. In the

foregoing the convention of signs is: clockwise moments and rotational displacements are positive; horizontal and vertical forces, displacements, and co-ordinate distances are positive when measured to the right or upwards.

The balancing movements at joint A induce movements at joint B. Similarly movements at joint B induce movements at joint A. Therefore the total displacement at a joint is the sum of the balancing movement (if any) at the joint and the induced movements created by balancing movements at other joints.

The deflection diagram for each member is obtained by proportion from the deflection diagrams produced by imposing unit displacements  $\phi$ ,  $\Delta$ , and  $\lambda$  in turn at each terminal. In some cases the diagram of the bending moments on each member due to end displacements is determined, and from it the deflection diagram is deduced by the moment-area method, taking into account the end displacements. The deflection diagrams are produced by the joint displacements caused by the imposed relative deformation and, for the member cut, they require to be combined, using the principle of superposition, with the deflection

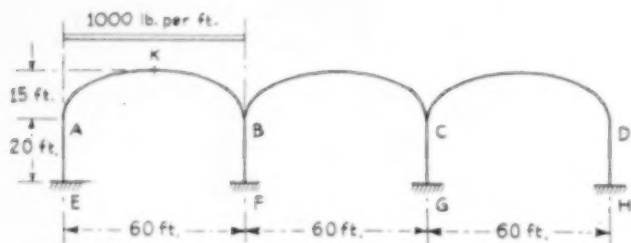


Fig. 3.

diagram produced by the indeterminate moment and forces resulting from the imposed deformation applied to the member when its ends are assumed to be fixed. For the member AB (Fig. 2), which contains the section at which unit relative displacement is applied, the final influence line is obtained by combining the deflection diagrams referred to in the foregoing. For all other members, however, the final influence line is the deflection diagram produced only by the joint displacements caused by the imposed relative deformation.

The influence line for the bending moment at a support or the reaction is simply a special case of the internal redundant being at one end of the member connected to the support.

The determination of an influence line comprises three steps, namely: (1). Structural analysis, to compile a table of unit displacements. (2). Fix the ends of the member containing the section for which the influence line is required. Impose unit relative deformation to the ends of a cut at this section, and obtain the deflection diagram produced by the indeterminate moment and forces corresponding to the imposed deformation. Release the fixed ends, and from the indeterminate moment and forces find the total joint displacements of all joints. (3). For each member obtain the deflection diagram caused by these total joint displacements and combine it with that obtained in (2). The result is the influence line required.

TABLE II.—RESULTS OF UNIT DISPLACEMENT.

JOINT A		JOINT B		JOINT C		JOINT D	
$\delta$	$\Delta$	$\delta$	$\Delta$	$\delta$	$\Delta$	$\delta$	$\Delta$
1.0	--	0.190	3.44	0.1237	1.972	0.1642	1.424
--	1.0	0.0228	0.550	0.0127	0.204	0.0108	0.1472
0.50	4.52	1.0	--	0.212	3.80	0.1928	2.63
0.0340	0.440	--	1.0	0.0245	0.364	0.0197	0.270
0.1928	2.63	0.212	3.80	1.0	--	0.30	4.52
0.0197	0.270	0.0245	0.364	--	1.0	0.0340	0.440

TABLE III.—TOTAL DISPLACEMENT OF JOINT RESULTING FROM UNIT RELATIVE ROTATION IMPOSED AT THE CROWN.

JOINT A		JOINT B		JOINT C		JOINT D	
$\delta$	$\Delta$	$\delta$	$\Delta$	$\delta$	$\Delta$	$\delta$	$\Delta$
0.1896	--	0.0560	0.652	0.0234	0.374	0.0197	0.270
--	2.973	0.0650	0.982	0.0378	0.606	0.0321	0.436
-0.0362	-0.545	-0.1204	--	-0.0257	-0.457	-0.0232	-0.317
-0.0794	-1.010	--	-2.392	-0.0339	-0.858	-0.0454	-0.622
0.0759	1.416	-0.0194	-0.668	-0.0204	-0.315	-0.0168	-0.231

### Example.

To illustrate the process, it is applied in the following to the three identical elliptical arches of 60-ft. span and 15-ft. rise shown in Fig. 3. The arches are continuous and are carried on piers 20 ft. high, the arches and piers being assumed to have the same cross section throughout. The influence line for the bending moment at the crown K of arch AB will be determined, and the magnitude of the bending moment at this section will then be calculated assuming there to be a uniformly-distributed load of 1000 lb. per linear foot on AB.

(1). The results of the first step in the calculations are given in Table No. II, which shows the displacements induced at all joints by imposing unit displacement at each joint in turn. The values of the displacement coefficients  $\phi_m$ ,  $\Delta_m$ , and  $\lambda_h$  are 4.44, 26.7, and 705 for joints A and D, and 3.15, 14.8, and 585 for joints B and C.

(2). Fix the ends A and B of the span; impose unit relative rotation at K. The centroidal moment and forces, from column (3) in Table No. I, are

$$m_0 = \frac{1}{74.8} = 0.0134; \quad h_0 = \frac{-(15 - 10.63)}{1300} = -0.00337; \quad \text{and } v_0 = 0.$$

Transferring the foregoing to A and B, the unbalanced moments and forces are

At A:  $M = -0.0134 + (0.00337 \times 10.63) = 0.0224$ ; and  $H = 0.00337$ .

At B:  $M = -0.0224$  and  $H = -0.00337$ .

From equations (6) the balancing movements are

$$\text{At A: } \phi = (0.0224 \times 4.44) + (0.00337 \times 26.7) = 0.1896.$$

$$\Delta = (0.0224 \times 26.7) + (0.00337 \times 705) = 2.973.$$

$$\text{At B: } \phi = (-0.0224 \times 3.15) - (0.00337 \times 14.8) = -0.1204.$$

$$\Delta = (-0.0224 \times 14.8) - (0.00337 \times 585) = -2.302.$$

By proportion from the unit displacements in Table No. II, the movements induced at other joints by these balancing movements can be calculated and the total joint displacements found as in Table No. III. For AB, the deflection diagram resulting from unit relative rotation at K, assuming the ends of the arch to be fixed, must be as in Fig. 4 (a). For the given load on AB, the fixed-end moment at A is 126,000 ft.-lb., and the fixed-end thrust is 36,000 lb. The vertical reaction

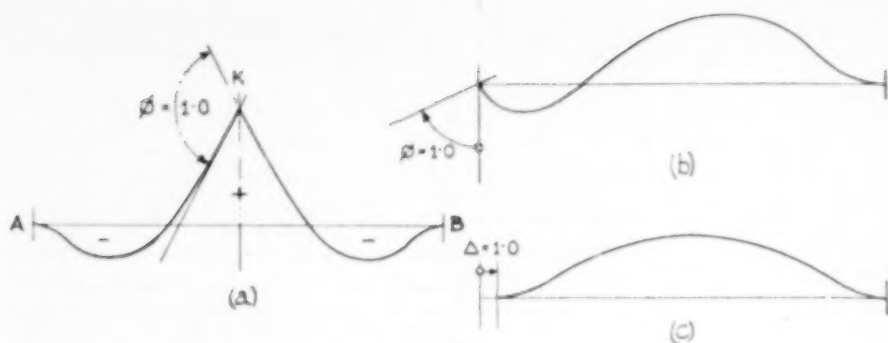


Fig. 4.

of the pier EA is 30,000 lb. Hence, the bending moment at K is 36,000 ft.-lb., so that the area of the deflection diagram in Fig. 4 (a) must be 36 sq. ft. per radian.

(3). For all spans, including AB, the deflection diagrams resulting from (i) unit relative joint rotation, and (ii) unit relative joint translation, are as shown in Fig. 4 (b) and (c) respectively. For the given load the area of the diagram for (i) is 126 sq. ft. per radian, and for (ii) 36 sq. ft. per foot.

From Table No. III, the relative rotation of A to B is  $(0.075 + 0.0194) = 0.0944$ , and the relative horizontal translation is  $(1.418 + 0.668) = 2.086$ . Applying these values to the areas of the deflection diagrams in Fig. 4 (b) and (c), the area of the actual deflection diagram of span AB is obtained, that is the area of the influence-line diagram for the bending moment at K due to joint rotations and translations. For the span AB the total area is

$$36 + (126 \times 0.0944) + (36 \times 2.086) = 122.9 \text{ sq. ft.}$$

For a uniformly-distributed load of 1000 lb. per foot on AB, the bending moment at K must therefore be  $122.9 \text{ sq. ft.} \times 1000 \text{ lb. per foot} = 122,900 \text{ ft.-lb.}$

### Prestressing an Arch Roof by Flat Jacks.

The concrete roof of a hull-testing tunnel at Toulouse is an arch slab prestressed transversely by cables anchored in cones of the Freyssinet type. Longitudinally the arch was pre-compressed by using flat jacks, which were placed in the transverse joints that occur every 100 ft. When pumped up, the flat jacks

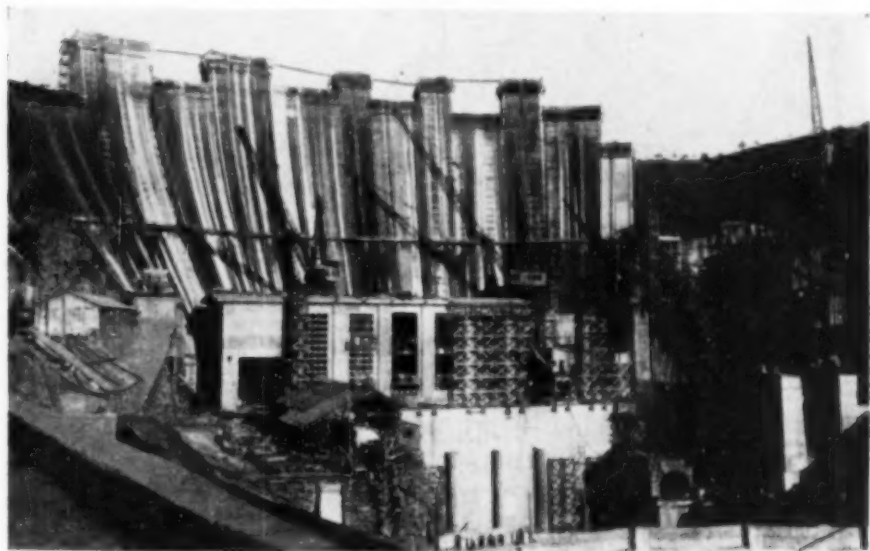
produce a longitudinal thrust which is resisted by the frictional resistance of the sections of the structure, the end section bearing against an abutment. The joints opened by the jacks are packed tight with mortar, and the jack is then deflated and removed. This type of jack is described on page 129, April, 1949.

## The Concreting Plant at the Castelo do Bode Dam, Portugal.

THE hydro-electric development of 88 miles of the river Zézere before it flows into the Tagus necessitates the formation of large reservoirs to regulate the very variable flow and to utilise the fall of about 900 ft. to the best advantage. The works will be in four sections, of which the Castelo do Bode and Cabril reservoirs, having a total capacity of about 1,350,000 acre-feet, are the largest. The power-stations will have a total installed capacity

bank and is designed for a maximum discharge of about 145,000 cu. ft. per second. The power station is at the foot of the dam and contains three vertical-reaction turbo-alternator sets, the rating of each set being 62,000 b.h.p. at an average net head of 262 ft. A plan and cross sections of the dam are given in *Fig. 2*.

Preliminary works included the construction of a road seven miles long and accommodation for workmen and en-



**Fig. 1.—Castelo do Bode Dam, November, 1949.**

of 434,000 h.p. and an annual output of 700,000,000 kw.-hours. The new works start with the Castelo do Bode reservoir as this provides the largest water storage and the greatest regulation of the flow of the river. This station will have an installed capacity of 186,000 h.p. and an annual output of 380,000,000 kw.-hours.

The Castelo do Bode dam (*Fig. 1*) is a thick arch structure 377 ft. high, and will give water storage of about 867,500 acre-feet in the form of a lake 38 miles long. The flood spillway is near the left-hand

engineers. A water-free site was formed by building concrete cofferdams above and below the site of the dam (*Fig. 3*). The upstream cofferdam diverts the course of the river through a tunnel, roughly parallel to the course of the river, having a cross-sectional area of 1023 sq. ft. and designed for a maximum discharge of 72,400 cu. ft. per second. Boring of the tunnel started in April, 1946, and in September, 1947, the site of the dam was dry. Towards the end of 1947 excavation by blasting for the dam and power station was started. Two

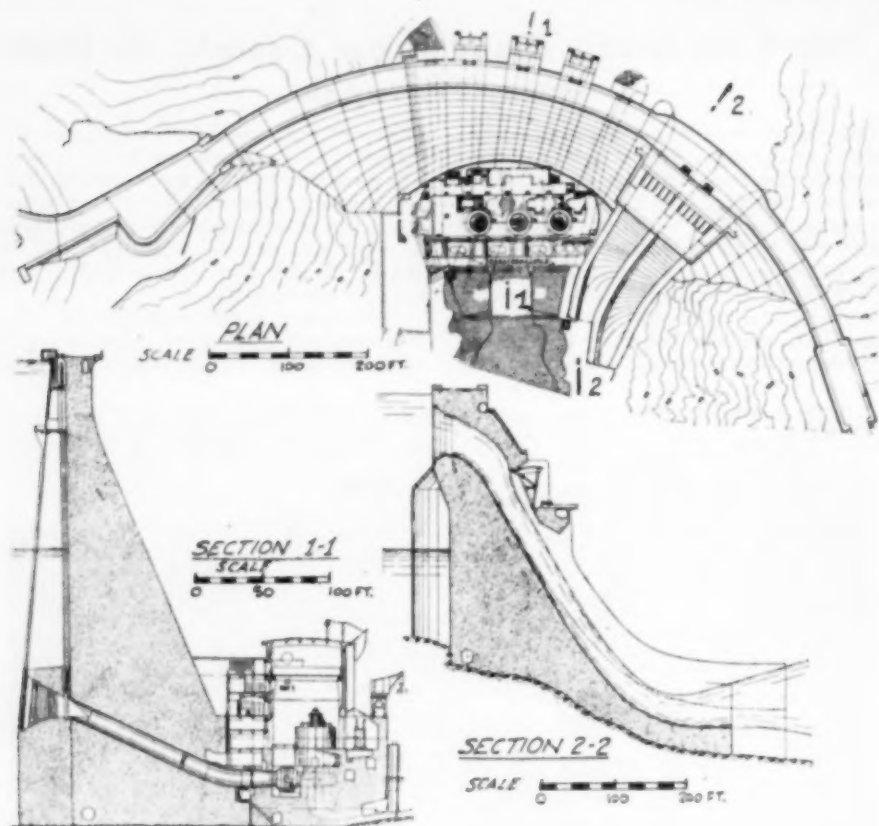


Fig. 2.—Plan and Cross Sections of Dam.

7-tons electric tower cranes were erected. Each crane has a jib 135 ft. long and a tower 110 ft. high, and is used for handling heavy materials, concrete (in the early stages of the work), machinery, and stores. There is also a 7-tons electric derrick crane with a 120-ft. jib which was used for the erection of the concrete-mixing plant. This crane can lift 7 tons at a radius of 90 ft. or 3 tons at a radius of 116 ft., and can be dismantled and transported for erection elsewhere.

The concrete-mixing plant consists of two installations, one close to the powerhouse and the other at a higher level. Concreting the foundation of the dam began in July, 1948, when only the lower plant was in use. The upper plant was in operation eight months later.

#### Concrete Materials.

About  $1\frac{1}{2}$  miles downstream from the dam, aggregate is dredged from the river-bed by six electrically-operated cableway excavators (Fig. 4). Four of these machines have a span of 820 ft., and two have a span of 655 ft. Each excavator has a  $1\frac{1}{2}$ -cu. yd. bucket and digs an average of 50 tons of material an hour. The bucket is lowered from the overhead cable on to the river-bed and dragged a short distance at slow speed until filled; the operator depends upon experience to judge when the bucket is filled. The full bucket is raised and brought to the bank, where it is automatically tipped and emptied into a hopper (Fig. 5). The empty bucket returns by gravity to the digging position.

The hopper discharges into storage bins (A in Fig. 4), from which tipping wagons are loaded. The wagons deliver the aggregate to 10-cu. yd. underground feed-hoppers (B) serving the washing and screening plant (C). A screen with 8-in. apertures is provided above the feed-hoppers to separate large pieces of stone. The material is conveyed from the feed-hoppers by belt apron-feeders which discharge on to three inclined 30-in. belt conveyors (D) supplying the washing and screening plant at a constant rate.

The washing and screening plant is in three sections each capable of dealing with

100 tons of aggregate per hour. The material passes first through a rotating cylindrical washer, and then through a rotary screen where the largest pieces of stone are removed. The smaller material falls on to two oscillating screens, having a combined vertical and forward motion, which grade the aggregate into six sizes, the graded material falling into storage bins below. The water from the first section of the screens contains sand which is recovered by a dewaterer, from which the sand is discharged into a storage bin below. The steel-framed storage bins below the washers and screens have a

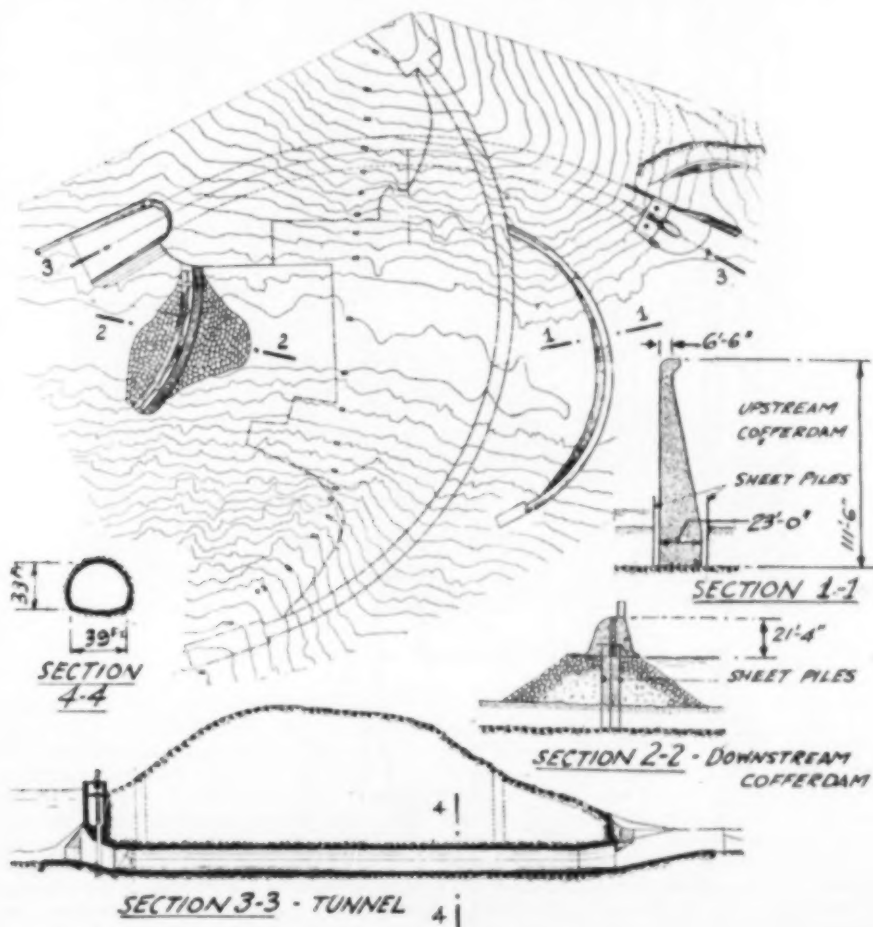


Fig. 3.—Diversion of River.

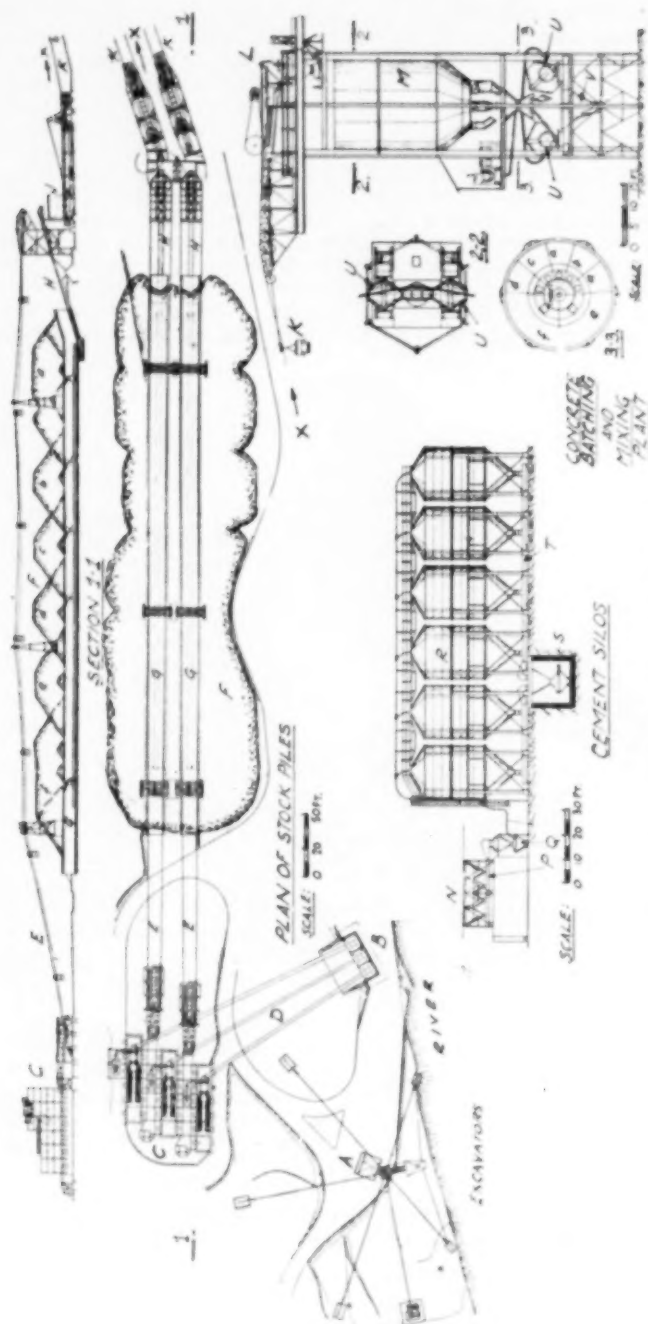


Fig. 4.—Materials and Concrete-making Plant.

capacity of 1800 tons, and each has outlets on both sides. From some of the outlets the aggregate is discharged into motor lorries, which carry the material to the lower mixing plant at the foot of the dam and which was used when constructing the cofferdams and the foundations of the main dam. The other outlets discharge the aggregate into buckets which are

because when the river is in flood excavation is impracticable, and because during storage the aggregates dry and about the same amount of water is added when mixing the concrete. The aggregate is stocked in the following sizes: (a) under 0.16 in.; (b) 0.16 in. to  $\frac{1}{8}$  in.; (c)  $\frac{1}{8}$  in. to 1 in.; (d) 1 in. to 2 in.; (e) 2 in. to 4 in.; and (f) 4 in. to 8 in.



Fig. 5.—Dredging Aggregate from River.

transported by two double-cable ropeways (E) to stock-piles (F) (Fig. 4). The two ropeways could transport 400 tons of aggregate an hour. The empty buckets, on reaching the storage bins, are released automatically from the hauling rope, and run by gravity on a shunt-rail to the loading position. When full they run by gravity to a locking frame, where the ropes are automatically gripped, and the buckets travel to storage piles for different grades of material. Storage is necessary

Conveyors (G in Fig. 4) in parallel tunnels (Fig. 6) under the stock-piles are fed from controlled outlets and pass the material to inclined belt-conveyors (H) which run to the top of storage bins (J) and deposit in these bins aggregate of the size needed at the mixing station at a particular time. The bins discharge into the buckets of two single-cable overhead ropeways (K) by which the aggregate is taken to a central concrete-mixing station more than  $1\frac{1}{4}$  miles distant. Each single-

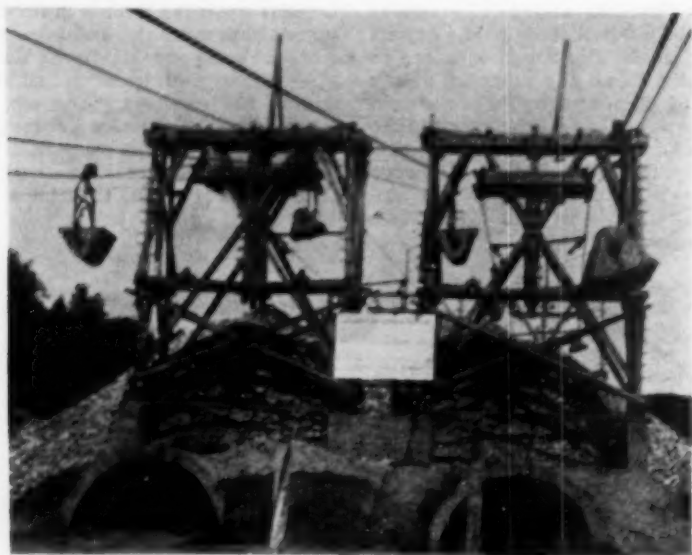


Fig. 6.—Conveyor Tunnels under Stockpiles.

cable ropeway has a capacity of 107 tons per hour. The trestles supporting the ropeways are of timber. The unloading terminals (L) are above the bunkers which serve the central mixing station, each of the two bins (M) of which has eight compartments and a capacity of 810 tons.

Seven compartments are for aggregates of different sizes (a) to (f), and the central one is for cement (Figs. 4 and 9).

The cement is brought to the site in containers on lorries from a railhead seven miles distant. On arrival the cement is discharged into a 50-tons hopper (N) from



Fig. 7.—Plant at Site of Dam.

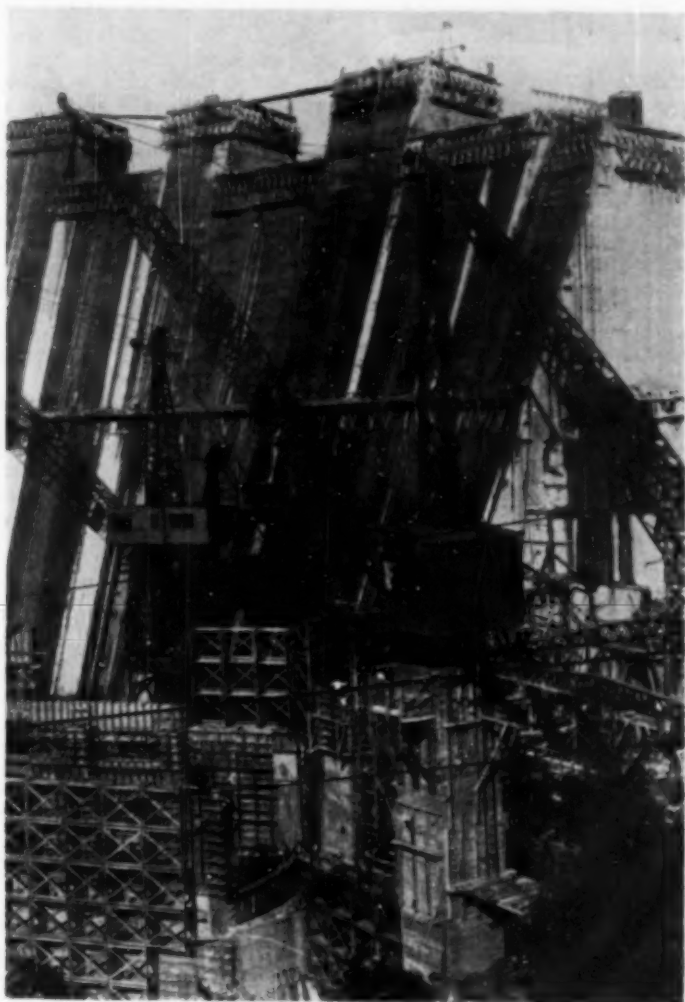


Fig. 8.—Tower and Derrick Cranes at Site of Dam.

which conveyors (P) carry it to an activator (Q) feeding six silos (R) (*Figs. 4 and 7*), having a total capacity of 3000 tons. An activator (S) and a conveyor (T) below the silos transfer the cement to the central mixing station.

#### Concrete Mixing and Distribution.

The aggregates and cement in the bunkers at the mixing station (*Figs. 4, 7, and 9*) are discharged through remote-

controlled pneumatically-operated gates into weighing hoppers. Other pneumatic controls discharge the contents of the hoppers into the mixers (U) below. There are four double-cone tilting-drum mixers, mounted in pairs. Each mixer has a capacity of 2 cu. yd. of mixed concrete. The water is measured by weight. The concrete is discharged into a central receiving hopper (V) below, and thence to 4-cu. yd. dumpers which carry it to an

unloading bay. The dumpers have a narrow automatic discharge-gate at the rear and approach the bay hopper foremost. At the edge of the bay a concrete kerb acts as a stop for the wheels of the dumper and trips a lever which automatically discharges the concrete into bottom-opening 4-cu. yd. buckets (Fig. 10), which, when filled, are picked up by the hoisting gear of an aerial cableway and raised at a speed of 250 ft. per minute. There are four aerial cableways each having a span of 1660 ft. and capable of lifting a load of 10 tons. The tail carriage of each cableway can be moved laterally so that the whole of the dam is covered. The length of the radial track on which this movement is made is 250 ft. for two of the cableways and 400 ft. for the other two. The fixed headmasts of two of the cableways are 100 ft. high and 150 ft. for the other two. The loaded bucket travels horizontally at the rate of 1000 ft.

per minute and is lowered at a rate of 250 ft. per minute. Sudden release of the contents, which might cause the bucket to jump upwards, is overcome by the use of manually-operated roller-gates which give a steady discharge of the concrete. As discharge takes place, upward movement is prevented by gradually lowering the hook from which the bucket is suspended so that the bucket is kept at the required height. The average amount of concrete placed by each cableway is 50 cu. yd. per hour. The cableways are also used for carrying the shuttering and removing rock after blasting. The rock is carried in flat-bottomed skips.

The lower of the concrete-mixing plants is equipped with drum mixers. The concrete is discharged into 2½-cu. yd. bottom-opening buckets which are transported on wagons running on narrow-gauge track on a bridge 135 ft. long to a position where they can be lifted by the tower cranes (Fig. 8). The slewing speed of these cranes when handling concrete buckets is 500 ft. per minute. Each crane lowers an empty bucket on to the wagon and lifts a full bucket off it. The bucket is hoisted at a speed of 80 ft. per minute. Empty buckets are lowered at 150 ft. per minute.

In the fourteen months to September, 1949, nearly 400,000 cu. yd. of concrete had been placed, which is about two-thirds of the total quantity required. High rates of concrete placing are being achieved, sometimes over 40,000 cu. yd. per month and 2100 cu. yd. daily have been placed.

For the body of the dam the concrete has a cement content of about 400 lb. per cubic yard, and an air-entraining agent enables it to be made very dry. Ice is added to the concrete if necessary to ensure that the temperature does not exceed 77° deg. F. under the most unfavourable conditions.

The main civil engineering works are expected to be completed this year. The dam is being constructed for the Hidro-Electrica do Zezere Co. The supply of the contractor's plant was co-ordinated by Messrs. Pauling & Co., Ltd., and was supplied by the following British firms: dragline excavators and aerial cableways, Messrs. John M. Henderson & Co., Ltd.; double- and single-cable ropeways, British Ropeway Engineering Co., Ltd.;



Fig. 9.—Central Concrete Mixing Station.

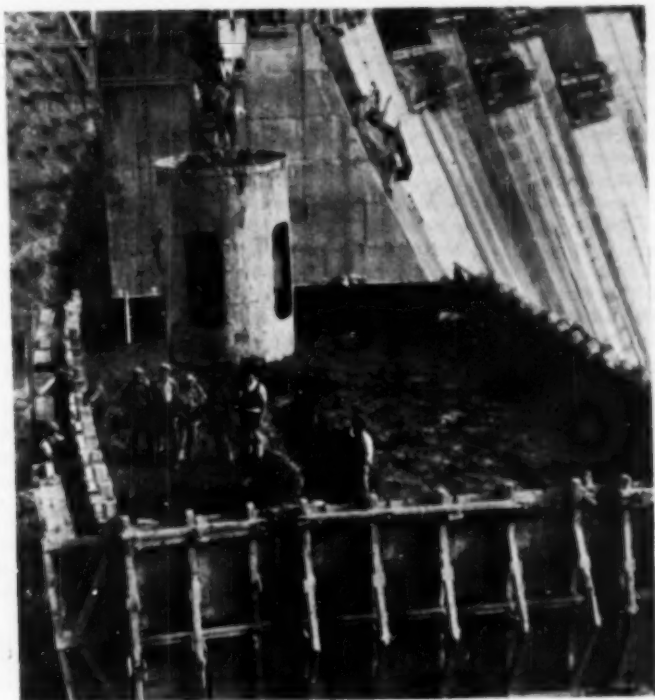


Fig. 10.—Placing Concrete from a 4-cu. yd. Bucket.

washing and screening plant, Messrs. Frederick Parker Ltd.; central-mixing station, cement silos, and concrete buckets, Blaw-Knox, Ltd.; concrete mixers, Messrs. Stothert & Pitt, Ltd.;

dumpers, Messrs. Aveling-Barford, Ltd.; tower cranes and connecting bridge, Messrs. Butters Bros. & Co., Ltd. The main contractors are Messrs. Monez da Maia e Vaz Guedes, Ltda.

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## Book Reviews.

"Concrete Products and Cast Stone." By H. L. Childs. Eighth Edition. (London: Concrete Publications Ltd. 1949. Price 2s. 6d.)

THE new edition of this book, which was first published in 1928 and remains almost the only book on the subject, is a revision of the previous edition (1940), the information having been brought up to date and amplified. The subject matter deals with the materials, proportioning, and making of dense and lightweight concrete, casting, consolidation, curing, surface finishes, reinforcement, the transport, storage, and methods of manufacture of different kinds of products, and the design and making of moulds of wood and other materials for a wide variety of products. New matter includes up-to-date information on the materials, chemicals, and processes employed in the manufacture of cellular ("gas" or "foamed") concrete; the fire-resistance and thermal properties of concrete; moulds for cylindrical products; the measurement of quantities; and illustrated descriptions of modern works using "production-line" methods of manufacture. All the methods of making concrete products that are described or illustrated are taken from practice, so that the information is of practical value to those who have only a few products to make or who are engaged in mass production.

"Winds, Waves, and Maritime Structures." By R. R. Miskin. (London: Charles Griffin & Co., Ltd. 1950. Price 25s.)

SOME books are of value because they bring together useful data from many sources, the collection being leavened by the author's experience and opinions. Such is this work of some two hundred pages which is useful because of the particulars of actual structures and information relating to natural phenomena connected with the weather and waves, some of the effects of which may not be known to engineers unfamiliar with nautical matters. Consideration of the pressure of waves against structures is based on theory and experiment. The difficulty of measuring such pressures on actual structures is obvious, but data is deduced from breakwaters that have suffered damage during storms of recorded violence. Some other matters dealt with are the foundations, arrangement and construction of breakwaters, sea-walls, and slipways, and dredging. In a space of

about 50 pages it cannot be expected that justice can be done to the design of jetties and dolphins and, apart from discussing the principles involved, the author gives detailed consideration to the impact of ships against marine structures, the behaviour of piles (based largely on experiments by the author on models), and the design of fenders. An omission, common to other books on marine structures, is a comprehensive practical method of determining the forces in the inclined and vertical piles of jetties.

"Hochfenschlacke." By P. F. Keil. (Düsseldorf: Verlag Stahlisen. 1949. Price D.M. 32.50.)

THIS publication of the German Iron and Steel Federation is a survey in the German language of the chemistry, manufacture, and applications of blastfurnace slag. The physical properties and methods of manufacture of the material and the chemistry of the silicates in the material are described. Consideration is given to slag sand (including granulation and glass content), glassy slag as an hydraulic binder, blastfurnace slag cements, and the use of slag as a lightweight building material, with special reference to heat and sound insulation. The use of slag in roads and civil engineering works is described, including details of water-bound and tarred macadam roads with and without surface treatment, concrete roads, water-resisting structures, and railways. The strength, workability, yield, and absorption of slag concrete are dealt with at length.

Although the applications of blastfurnace slag and its products to the technique of road construction are fully covered by British Standard Specifications, this book, which contains 346 pages, 107 good illustrations, and 72 tables, should be of interest to specialists on account of its comprehensiveness.

"Bestimmungen des Deutschen Ausschusses für Stahlbeton." By B. Wedler. (Berlin: Wilhelm Ernst & Sohn. 1949. Price D.M. 5.)

THIS is the third edition of the German regulations for the design and construction of reinforced concrete. Except for minor revisions this edition differs from the issue of 1947 only by the addition of regulations for the restoration of damaged reinforced concrete buildings and of rules for the construction of lightweight concrete walls.

## Design of Reinforced Concrete Members in accordance with the British Standard Code\*—IV.

By CHAS. E. REYNOLDS, B.Sc., A.M.Inst.C.E.

### Columns Subjected to Bending.

THE external effects on a column (*Fig. 1a*) may include one or more of the following.

(1).—A number of loads  $P_1$  to  $P_n$  acting parallel to the axis of the column at known distances  $g_1$  to  $g_n$  from, say, the centre of the reinforcement  $A_T$  nearer to the edge at which the compressive stress is lowest or at which tensile stress is induced. If the loads act downwards they are positive, and, if they act to the right of  $A_T$  (*Fig. 1b*),  $g$  is also considered to be positive. The loads may be the loads from a crane-beam or other weight on a corbel on the side of the column, or from a longitudinal beam bearing centrally or eccentrically on the column, or from a column above that being considered, or the weight of the column itself.

(2).—A moment  $M$  (assumed to be positive if it acts in a clockwise direction as in *Fig. 1b*). This moment is the algebraic sum of moments imposed by transverse beams built monolithically with the column, or by forces, such as those due to wind, acting transversely to the axis of the column.

(3).—If the moment  $M$  is partly or entirely due to the restraint of beams built monolithically with the column it will be accompanied by a vertical load  $P_a$ , the position of the line of action of which is indefinite, and it is proper to consider that  $P_a$  acts at the centroid  $X$  (*Figs. 1a* and *b*) of the stressed area of the column, since only in this position ( $g = \bar{x} = x_1 D$ ) is  $P_a$  equivalent to an axial load producing no additional moment on the column, the entire moment effect being included in  $M$ .

The following analyses of columns apply to the common case of the imposed loads and moments acting in the plane containing one of the axes of the column.

There are two conditions of stress produced by eccentric loads and moments, namely: Case I.—When the stresses are wholly compressive, the stress in the reinforcement being  $m$ -times that in the surrounding concrete; in accordance with the Code,  $m = 15$ . Case II.—When tensile and compressive stresses are induced simultaneously; in accordance with the Code, the tensile resistance of the concrete is neglected and rectilinear variation of strain is assumed, with  $m = 15$ . Other requirements of the Code are described later.

ECCENTRICITY.—The loads  $P_1$  to  $P_n$  can be replaced by a load  $N_e$  equal to  $\sum_i P_i$  and acting at a distance  $e \left( = e_1 D = \frac{\sum_i P_i g_i}{\sum_i P_i} \right)$  from  $A_T$  as in *Fig. 1c*. The moment  $M$  can be replaced by two opposite forces each equal to the net load  $N$  on the column; one of the forces acts at  $X$  and the other  $\frac{M}{N}$  therefrom (*Fig. 1c*). The net load  $N$  on the column is  $N_e + P_a + N - N$ , that is  $N_e + P_a$ . The moment about  $A_T$  of the loads is  $-N\bar{x} + N \left( \bar{x} + \frac{M}{N} \right) + P_a \bar{x} + N e_1 D$ , that is

\* Concluded from January, February, and April, 1950.

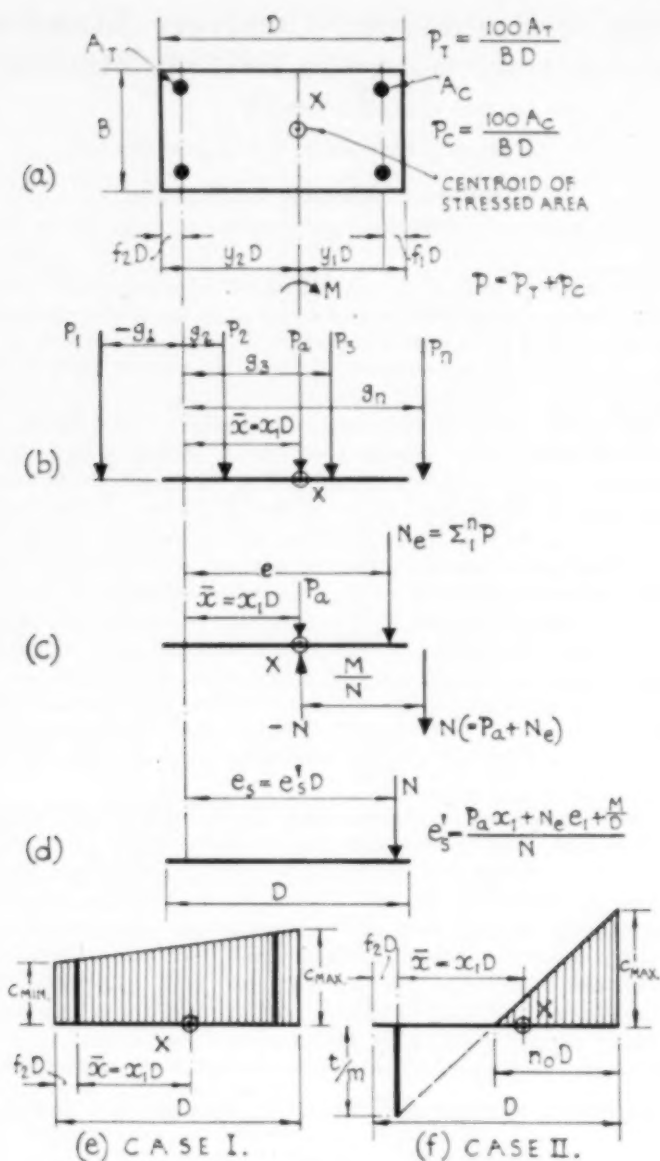


Fig. 1.

$P_a \bar{x} + N_e e_1 D + M$ . Therefore the loads and moment can be replaced by a single load  $N$  acting at a distance  $e_s (= e_1 D)$  from  $A_T$ , as in Fig. 1d, such that

$$N = P_a + N_e \text{ and } e_s = \frac{1}{N} \left( P_a x_1 + N_e e_1 + \frac{M}{D} \right).$$

In a given case  $e'_s$  can be calculated, since the terms in this expression can be numerically evaluated, with the exception of  $x_1$  in Case II where  $e'_s$  can be evaluated in the form  $e'_s = ux_1 + v$ , where  $u$  and  $v$  are numerical constants. If in Case I  $e'_s$  equals  $x_1$ , uniform strain is produced on the entire cross section of the column; uniform strain is the condition of axial load dealt with in the previous article.

CASE I.—COMPRESSIVE STRESSES ONLY.—In this case (*Fig. 1 c*) the whole cross-sectional area  $A$  of the column is resistant and the common formulæ, modified to suit the notation in this article, for the maximum and minimum compressive stresses apply, the eccentricity and moment of inertia  $I_x$  being measured from the centroid  $X$  of the stressed area. Therefore

$$c_{\max.} = N \left[ \frac{1}{A} + \frac{(e_s - \bar{x})y_1 D}{I_x} \right] \text{ and } c_{\min.} = N \left[ \frac{1}{A} - \frac{(e_s - \bar{x})y_2 D}{I_x} \right],$$

in which

$$A = A_1 BD, \text{ where } A_1 = 1 + 0.01(m-1)(p_T + p_c)$$

$$\bar{x} = x_1 D, \text{ where } x_1 = [(0.5 - f_2) + 0.01 p_c (m-1)(1 - f_1 - f_2)] \div A_1,$$

$$y_1 = 1 - f_2 - x_1; \quad y_2 = x_1 + f_2; \quad e_s = e'_s D.$$

$$I_x = I_1 BD^3, \text{ where } I_1 = \frac{y_1^3 + y_2^3}{3} + 0.01(m-1)[p_c(y_1 - f_1)^3 + p_T x_1^3].$$

Therefore in the general case

$$c_{\max.} = \frac{N}{BD} \left[ \frac{1}{A_1} + \frac{(e'_s - x_1)y_1}{I_1} \right] \text{ and } c_{\min.} = \frac{N}{BD} \left[ \frac{1}{A_1} - \frac{(e'_s - x_1)y_2}{I_1} \right].$$

For the special, and common, case of  $m = 15$ ,  $A_c = A_T$  (and  $p = p_c + p_T$ ), and  $f_1 = f_2 = 0.1$ ;  $A_1 = 1 + 0.14p$ ;  $x_1 = 0.4$  (that is the centroid of the stressed area is on the centre-line of the section);  $y_1 = y_2 = 0.5$ ; and  $I_1 = \frac{1}{12} + 0.0224p$ . Therefore

$$\frac{c_{\max.}}{c_{\min.}} > \frac{N}{BD} \left[ \frac{1}{1 + 0.14p} \pm \frac{6(e'_s - 0.4)}{1 + 0.27p} \right].$$

The foregoing expression for  $c_{\max.}$  can be written  $c_{\max.} = Q_1 \frac{N}{BD}$ , where

$$Q_1 = \frac{1}{1 + 0.14p} + \frac{6(e'_s - 0.4)}{1 + 0.27p}.$$

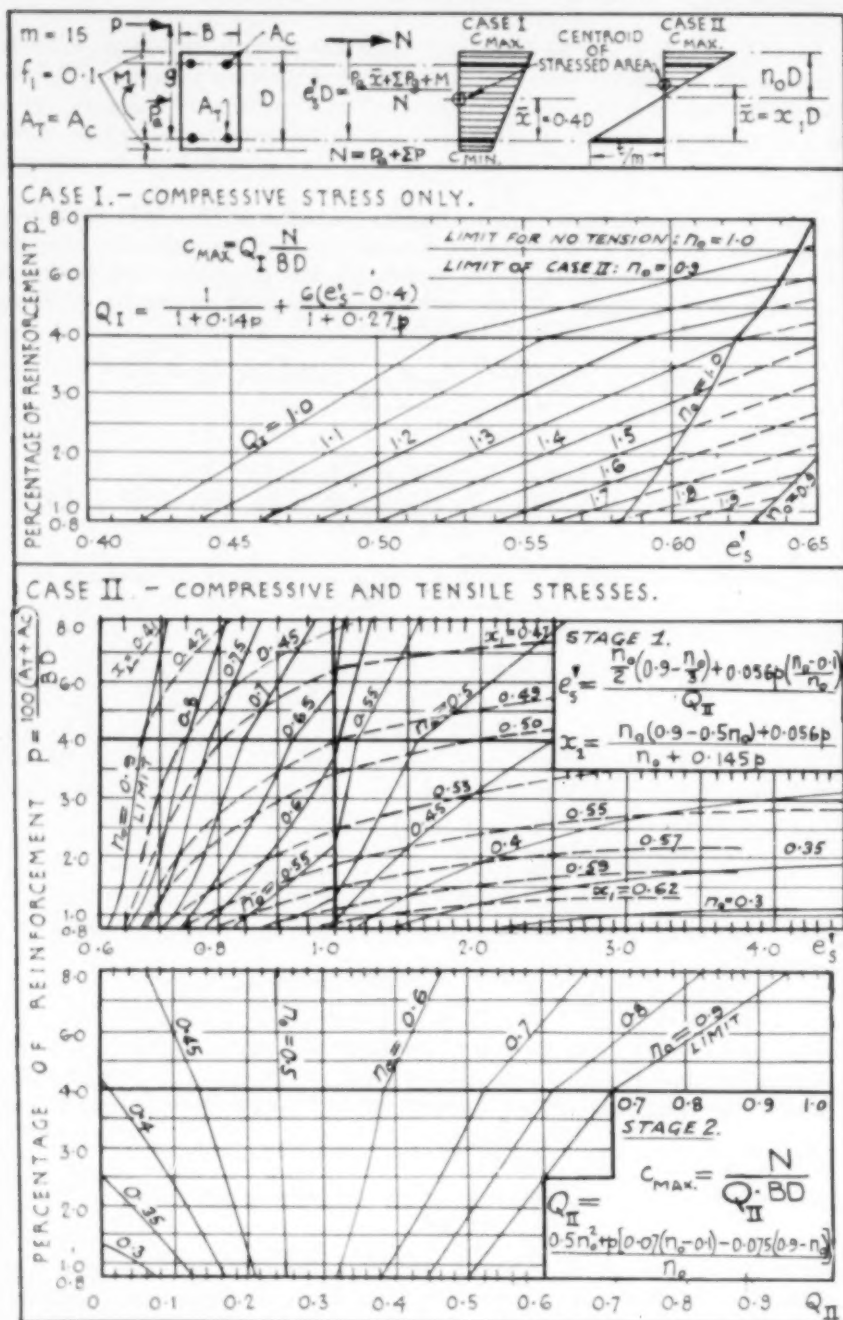
The curves in the upper part of *Table No. 7* give values of  $Q_1$  (from which  $c_{\max.}$  can be calculated) for known values of  $p$  and  $e'_s$ ; or values of  $p$  (from which the amount of reinforcement can be determined) for known values of  $e'_s$  and  $Q_1$  which in this case is  $\frac{BDc_{\text{permissible}}}{N}$ .

The minimum compressive stress in the concrete is given by

$$c_{\min.} = \frac{2N}{(1 + 0.14p)BD} - c_{\max.}$$

CASE II.—TENSILE AND COMPRESSIVE STRESSES.—The basic formulæ for this case are derived from the two axioms that the algebraic sum of the internal and external forces acting on any cross section of a column is zero, and that the

TABLE 7.—COLUMNS SUBJECTED TO BENDING.



NOTE.—Table No. 7 is applicable not only to columns designed in accordance with the British Standard Code, but to any rectangular member the design of which is based on  $m = 15$ ,  $f_1 = f_2 = 0.1$ , and  $A_C = A_T$ .

algebraic sum of external moments and moments about any axis (say the centre of  $A_T$ ) of internal and external forces is also zero. Evaluation in terms of the dimensions and stresses in Fig. 1a and f gives respectively the two general equations:

$$\frac{N}{BD} = c_{\max} \left[ \frac{n_0}{2} + 0.01 p_c (m-1) \left( \frac{n_0 - f_1}{n_0} \right) - 0.01 p_t m \left( \frac{1 - n_0 - f_1}{n_0} \right) \right]$$

= say,  $c_{\max} Q_{II}$ .

$$\frac{N e'_s}{BD} = c_{\max} \left[ \frac{n_0}{2} \left( 1 - \frac{n_0}{3} - f_1 \right) + 0.01 p_c (m-1) \left( \frac{n_0 - f_1}{n_0} \right) (1 - f_1 - f_1) \right].$$

For the special, and common, case of  $m = 15$ ,  $A_c = A_T$  (and  $p = p_c + p_t$ ), and  $f_1 = f_2 = 0.1$ , the foregoing equations can be transposed to

$$c_{\max} = \frac{N}{Q_{II} \cdot BD}$$

$$Q_{II} = \frac{0.5 n^2 + p [0.07 (n_0 - 0.1) - 0.075 (0.9 - n_0)]}{n_0}$$

$$e'_s = \left[ \frac{n_0}{2} \left( 0.9 - \frac{n_0}{3} \right) + 0.056 p \left( \frac{n_0 - 0.1}{n_0} \right) \right] \frac{1}{Q_{II}}$$

The unknown quantity,  $n_0$ , is not easy to evaluate, but the curves in the middle and lower parts of Table No. 7 enable most problems in Case II to be expeditiously solved. The procedure is to find from the curves in stage 1 the value of  $n_0$  for known values of  $e'_s$  and  $p$ ; then from the curves in stage 2 the value of  $Q_{II}$  is found for this value of  $n_0$  and the known value of  $p$ . The compressive stress in the concrete is calculated from  $c_{\max} = \frac{N}{Q_{II} BD}$ , and the tensile stress in the reinforcement is calculated from  $t = 15 c_{\max} \left( \frac{0.9}{n_0} - 1 \right)$ .

The foregoing procedure is straightforward if  $P_a$  is zero since  $e'_s$  can then be evaluated directly. If there is a load  $P_a$ , the unknown in the expression for  $e'_s$  is  $x_1$ , and from the given data  $e'_s$  can only be evaluated in the form of  $e'_s = u x_1 + v$  as before stated. In the general case of Case II,

$$x_1 = \frac{(m-1) p_c (1 - f_2 - f_1) + 100 n_0 (1 - f_2 - 0.5 n_0)}{m p_T + (m-1) p_c + 100 n_0}$$

For the special and common case (Table No. 7) of Case II,

$$x_1 = \frac{0.056 p + n_0 (0.9 - 0.5 n_0)}{0.145 p + n_0}$$

and the procedure when  $P_a$  is not zero is to assume a value of  $x_1$  thereby giving a numerical value to  $e'_s$ . (Note that  $x_1$  cannot be less than 0.4 and is not likely to exceed 0.65.) Enter the curves in stage 1 with this value of  $e'_s$  and the known value of  $p$  and note the value of  $x_1$  as indicated by the dash-line curves. If this differs from that assumed select a second and further values (two trials are generally sufficient). Then read off the corresponding value of  $n_0$  and proceed to stage 2 as before.

PROBLEMS BETWEEN CASES I AND II.—The limit of application of Case I is

when  $c_{\min.} = 0$ , that is when  $\frac{I}{A_1} = \left( \frac{c_s - x_1}{I_1} \right) y_2$ , in which case the neutral axis is at the edge of the section. The limit of application of Case II is when  $n_0 = 1 - f_1 (= 0.9$  in the special case), in which case  $t = 0$ , the neutral axis is at  $A_T$ , and the tensile resistance of the concrete between  $A_T$  and the adjacent edge is neglected. If a small tensile stress is permitted in the concrete, the curves and formulæ for Case I can be used for cases between I and II; in the special case the tensile stress will not exceed one-ninth of  $c_{\max.}$  and is given by the negative result of solving the formula for  $c_{\min.}$ . If, as in accordance with the Code, the tensile resistance of the concrete must be neglected, the maximum compressive stress in border-line cases (that is when  $c_{\min.}$  is negative) will be increased to approximately  $\frac{N}{0.9Q_1BD}$ .

OTHER COVER RATIOS.—The curves in Table No. 7 are limited to cover ratios of one-tenth, that is  $f_1 = 0.1$ , but approximate adjustments can be made for other ratios. If  $f_1$  is less than 0.1, the value of  $c_{\max.}$  calculated from the curves will be slightly higher than the actual stress; therefore Table No. 7 can be used to determine a value which will not be exceeded. If  $f_1$  is greater than 0.1, the stress will be higher than that calculated from the curves, and a fair approximation can be made if for  $D$  a value  $D_1$  equal to  $(1.1 - f_1)D$  is substituted. Generally, if  $f_1$  exceeds 0.10 and Case I applies, it is more accurate and almost as simple to use the formula for  $c_{\max.}$  instead of Table No. 7.

REQUIREMENTS OF THE CODE.—The curves in Table No. 7 apply to designs in accordance with the Code (if adjusted as described in the foregoing for other values of  $f_1$ ) as they are calculated for  $m = 15$  and are therefore applicable to all mixtures of concrete. The Code requires the amount of reinforcement in a column to be not less than 0.8 per cent. and not more than 8 per cent., and this is the range of  $p$  given in Table No. 7. Nevertheless it is seldom economical to provide the higher percentages.

The permissible compressive stresses in a member subjected to bending and compression are the same as in beams, but it is necessary to ensure that the load on the column, assuming the moment not to operate, does not produce stresses exceeding those permitted for axially-loaded columns. The tensile stress in mild steel bars in members subjected to bending must not exceed 18,000 lb. per square inch. The permissible stresses must be reduced for long columns subjected to bending in the same proportions as for long axially-loaded columns, but the reduction does not apply to the stresses in the part of the column within one-eighth of the length of the column from either end.

#### EXAMPLES OF COLUMNS SUBJECTED TO BENDING.

In the following examples, it is assumed that the dimensions of a column of 1 : 1 : 2 vibrated concrete ( $m = 15$ ) are  $D = 20$  in. and  $B = 12$  in. The reinforcement is three 1-in. bars at each of the two narrow edges, and the cover of concrete is  $1\frac{1}{2}$  in. Hence  $f_1 = f_2 = \frac{3}{20} = 0.15$ , and Table No. 7 can be used to calculate the maximum stresses.  $p = \frac{100 \times 4.71}{20 \times 12} = 2$  per cent.

(i) A bending moment  $M$  of 240,000 in.-lb. acts in conjunction with a load

$P_a$  of 120,000 lb. Therefore  $N = 120,000$  lb. Assuming Case I to apply,  $\bar{x} = 0.4D = 8$  in. and  $e'_s = \frac{(120,000 \times 8) + 240,000}{120,000 \times 20} = 0.5$ , which falls in Case I as assumed. From Table No. 7, with  $p = 2$  and  $e'_s = 0.5$ ,  $Q_1 = 1.17$ ; therefore

$$c_{\max.} = \frac{1.17 \times 120,000}{12 \times 20} = 585 \text{ lb. per square inch.}$$

(ii) A bending moment  $M$  of 720,000 in.-lb. acts in conjunction with a load  $P_a$  of 120,000 lb. As this is probably in Case II, assume  $x_1$  to be 0.5.

$$e'_s = \frac{(120,000 \times 0.5 \times 20) + 720,000}{120,000 \times 20} = 0.8,$$

which is Case II as assumed. From the curves for stage 1 (Table No. 7), for  $e'_s = 0.8$  and  $p = 2$  per cent.,  $x_1$  is about 0.51 which is near enough to the assumed value, and for the foregoing values of  $e'_s$  and  $p$ ,  $n_0 = 0.67$ . From the curves for stage 2, with  $n_0 = 0.67$  and  $p = 2$  per cent.,  $Q_{11}$  is about 0.41 and

$$c_{\max.} = \frac{120,000}{0.41 \times 12 \times 20} = 1225 \text{ lb. per square inch.}$$

(If the stress is calculated on the common assumption that the eccentricity  $\frac{M}{N}$  is measured from the centre-line of the section,  $c_{\max.}$  is 980 lb. per square inch.)

(iii) The column is subjected to a load  $P$  of 20,000 lb. acting 4 in. outside one edge.

Therefore  $N = P = 20,000$  lb. and  $g = 4 + 18 = 22$  in.  $= e'_s D$ . Therefore  $e'_s = 1.1$  (Case II). From stage 1 (Table No. 7), for  $e'_s = 1.1$  and  $p = 2$  per cent.,  $n_0 = 0.51$ . From stage 2,  $Q_{11} = 0.27$ . Therefore

$$c_{\max.} = \frac{20,000}{0.27 \times 12 \times 20} = 310 \text{ lb. per square inch.}$$

(iv) The column is subjected to the effects specified in (i) and (ii) at the same time, assuming both moments ( $M$  and  $Pg$ ) to be positive.

$$N = 120,000 + 20,000 = 140,000 \text{ lb.}$$

Assume  $x_1 = 0.5$ ; then

$$e'_s = \frac{(120,000 \times 0.5 \times 20) + (20,000 \times 22) + 720,000}{140,000 \times 20} = 0.84.$$

For  $e'_s = 0.84$  and  $p = 2$  per cent., from Table No. 7,  $x_1$  is a little less than 0.52; therefore, if  $e'_s$  is recalculated assuming  $x_1 = 0.52$ ,  $e'_s$  is 0.86 and, from the curves,  $n_0$  is 0.62 and  $Q_{11}$  is 0.37. Therefore

$$c_{\max.} = \frac{140,000}{0.37 \times 12 \times 20} = 1578 \text{ lb. per square inch,}$$

which does not exceed the stress of 1650 lb. per square inch permitted by the Code in vibrated 1:1:2 concrete. The requirement that the stress assuming the total load to act axially must not exceed that permitted in axially-loaded columns must be complied with, and is

$$\frac{140,000}{(14 \times 4.71) + (12 \times 20)} = 458 \text{ lb. per square inch,}$$

compared with the permissible stress of 1140 lb. + 10 per cent., that is 1254 lb. per square inch.

## Concrete Piles at Beckton Gas Works.

THE extensions and reconstruction now proceeding at the Beckton Gas Works of The North Thames Gas Board (successors to the Gas Light & Coke Co.) and at the adjacent by-products works require piled foundations, as do most structures at these works. Some interesting features of the piles being driven, or recently driven, are the use of high-alumina cement and prestressed concrete. (Ordinary piles, of which there are several thousand up to 70 ft. in length, under the coke-oven plant at Beckton were described in this journal, September, 1931.)

### High-Alumina Cement Concrete Piles.

The ground at the by-products works comprises a deposit of chemical refuse and acid soils several feet thick overlying ballast. The precast concrete piles for the foundations of new buildings, including the Welfare block and the Mills-Packard plant, are made with high-alumina cement to resist attack by the acids in the ground. Most of these piles (*Fig. 1*) are 10 in. square, but a few are 12 in. and 14 in. square. The main reinforcement in each pile comprises four bars the diameters of which are  $\frac{1}{2}$  in. in 10-in. piles,  $\frac{3}{4}$  in. in 12-in. piles, and 1 in. in 14-in. piles. The ends of the bars do not bear on the shoe. The shoes are of cast iron, and they weigh 30 lb. for the 10-in. and 12-in. piles and 40 lb. for the 14-in. piles. The mixture of the concrete is 3 cwt. of high-alumina cement,  $8\frac{1}{2}$  cu. ft. of sand, and  $13\frac{1}{4}$  cu. ft. of shingle. The moulds were removed from six to eight hours after casting, and the concrete was sprayed with water for at least two days after stripping. The piles were cast at two works about five miles and ten miles respectively from Beckton and were driven at various ages up to six weeks.

In the foundation of the Welfare block there are about 170 piles, each 30 ft. long and bearing on ballast about 22 ft. below the surface. There are 101 piles 10 in. square, the greatest working load on one pile being 45 tons; 56 piles 12 in. square, load 57½ tons; and 16 piles 14 in. square, load 70 tons. Before driving the piles test piles, two of each of the three sizes, were driven and loaded. Because of the nature of the ground and the small

number of piles, light and easily-handled piling plant was used and included a 50-cwt. winch, a frame with 40-ft. leaders, and 1-ton and 2-ton drop-hammers with trigger release enabling the effects of free drops to be studied. The weight of the hammer and the height of the drop were such that high stresses in the head of the pile during driving were avoided. The penetration and number of blows throughout the driving of each test pile were recorded. When driven the piles were tested by loading. The concrete was stripped from the heads of the piles and the heads remade with a plane horizontal surface to receive a steel plate on which a 200-ton hydraulic jack was placed. The reaction from the jack was transmitted to three large steel beams which transmitted the load to a platform of twenty 10-in. by 6-in. steel beams on which rails were stacked. A steel clamp with horizontal projecting limbs was fixed to the head of the pile. A scale was held on the limbs and observed through a dumpy level. Observations were taken on each limb at each increment of load to ensure a true mean reading of the settlement. The ultimate load on each of the 10-in. piles, 30 ft. long, was 90 tons; on one 12-in. pile, 30 ft. long, 115 tons; on one 14-in. pile, 30 ft. long, 140 tons; and on one 12-in. and one 14-in. pile, both 35 ft. long, 150 tons.

No load tests were made on the piles for the Mills-Packard sulphuric-acid plant as the information and experience gained when driving the piles for the Welfare block were considered sufficient. The site of this plant was a spent-acid pit used for depositing chemical refuse about forty years ago. The lime-sludge deposit, which would virulently attack Portland cement concrete, extends to about 6 ft. from the surface, and below is silt which extends to about 25 ft. below the surface at which depth ballast is encountered. The piles are 28 ft. long, penetrate about 2 ft. into the ballast, and are driven by a 2-ton winch-operated drop-hammer falling 2 ft. There are 208 10-in. piles on this site and twenty 12-in. piles. The greatest working loads are the same as on the 10-in. and 12-in. piles under the Welfare block. Two trial piles 30 ft. long were driven first, and from the results the length of 28 ft.

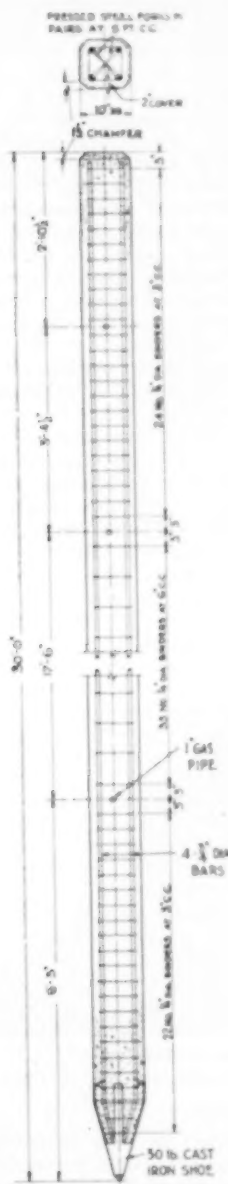


Fig. 1.—High-alumina Cement Concrete Pile.

was determined for subsequent piles. The piles for this plant were driven to a set of three blows for a penetration of 1 in. for the 10-in. piles, and four blows for the 12-in. piles plus a further penetration of 6 in. for which the number of blows for each inch was not to be less than the minimum specified. Owing to obstacles buried in the chemical deposit, the driving of each pile was preceded by driving a steel needle a few feet into the ground. The needle, which was made of fabricated steel sections, broke up any obstacles thereby facilitating, when the needle was withdrawn, the driving of the permanent piles.

The consulting engineers for the reconstruction of the by-products works are Messrs. Brian Colquhoun & Partners. The piles for the Welfare block were driven by the main contractors, Taylor Woodrow Construction, Ltd., and were made by Stent Precast Concrete, Ltd. The piles for the Mills-Packard plant were made and driven by Messrs. W. & C. French, Ltd.

#### Prestressed Concrete Piles.

The prestressed concrete piles (Fig. 2) are 40 ft. long and 10 in. square. Prestressed concrete was adopted in order that the piles would be more likely to be free from cracks and therefore less liable to be attacked by acids in the soil, as they penetrate gas-works lime and other deposits before reaching ballast at various depths. The average depth at which ballast is encountered on the site of the coke-grading plant is about 25 ft., but the piles penetrate several feet into this stratum. With prestressed concrete piles a smaller cross section is required owing to the high strength of the concrete, so that the weight of the pile and consequently the ratio of the weight of pile to the weight of the hammer are reduced; in the present case this ratio is about unity if a 2-tons hammer is used. Also, the use of prestressed piles results in a saving of about 70 per cent. of the weight of steel compared with reinforced concrete. The concrete is mixed in the proportions of 1 : 1½ : 3 and consolidated by vibration, the compressive strength specified being 5500 lb. per square inch at the time the stretching force is released and 7500 lb. per square inch at 28 days. The pre-compression in the concrete is about

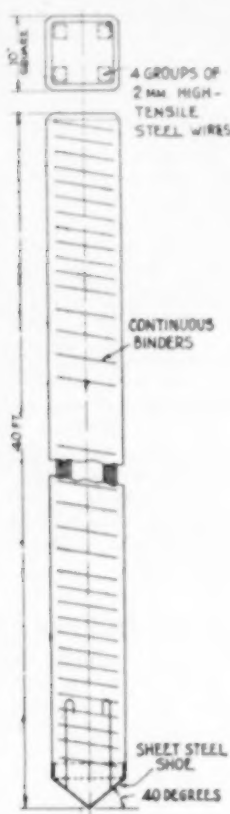


Fig. 2.—Prestressed Concrete Pile.

800 lb. per square inch and is induced by four groups of wires, each group comprising twenty wires of 2 mm. diameter and stretched until the tensile stress is 90 tons per square inch. The shoes are of steel plate forming a fairly blunt end. Toggle holes are provided for lifting. Owing to the precompression, single-point lifting is used for a 40-ft. pile.

Three trial piles, which are thought to be the first prestressed concrete piles to be driven in this country, were driven at the site of a new gasholder at Stanford-le-Hope. Since the trial piles, which are 10 in. square and up to 35 ft. long, are not part of the permanent structure, it was possible to test them to destruction. The driving was very severe. The weight of the hammer was 2 tons 16 cwt. and a drop of 4 ft. to 6 ft. was necessary to cause the head of the pile to shatter when the set was consistently  $\frac{1}{2}$  in. for ten blows. The age of the piles when driven was from seven days to two months. In each case the required set was obtained about 17 ft. below the surface.

In the foundations for a new substation at Beckton, thirty-six prestressed concrete piles are provided. The piles for the substation were driven to sets of ten blows to  $\frac{1}{2}$  in. to 1 in. when from seven to ten days old, the toes being about 35 ft. below the surface. A pile on another site at Beckton was subjected to a static test load. This pile was driven to a set of 2 in. for ten blows. A concrete head was cast on top of the pile and a balanced frame of steel joists erected thereon. The load, which comprised old concrete pile-heads, was 40 tons before any settlement was recorded; when it was increased to 75 tons the settlement was about 0.3 in. and remained constant throughout the three days during which this load was maintained.

The prestressed concrete piles were made by the Concrete Development Co., Ltd. The piles at Stanford-le-Hope were driven by Concrete Piling, Ltd. The piles for the substation at Beckton were driven by Messrs. John Shelbourne & Co., Ltd., for the main contractors, Messrs. Arup & Arup Ltd.

The foregoing works were designed and constructed under the supervision of Dr. J. Burns, Chief Engineer of the North Thames Gas Board.

### Prestressed Concrete Piles.

PRESTRESSED concrete piles, similar to those described in the preceding article, but 30 ft. and 40 ft. long are also being driven at Grays, Essex. At this site there are 193 10-in. square piles which are driven when 28 days old to an average set of  $\frac{1}{2}$  in. for ten blows. The driving of 125 prestressed concrete piles 12 in. square and

50 ft. long has recently been completed at a site on the Clyde for the Fairfield Shipbuilding and Engineering Co. All the foregoing piles were made by the Concrete Development Co., Ltd., those at Grays were driven by Messrs. W. & C. French, Ltd., and on the Clyde by Messrs. Holland & Hannen and Cubitts, Ltd.

## Construction with Moving Forms.—III\*

By L. E. HUNTER, M.Sc., A.M.Inst.C.E.

### The Deck.

THE deck, or platform, which moves upwards with the forms generally comprises wooden boards covering the spaces between the walls of the structure and supported on wooden joists, the ends of which bear on the inner walings of the forms as seen in *Figs. 10 and 13*. Access to the deck from below is usually

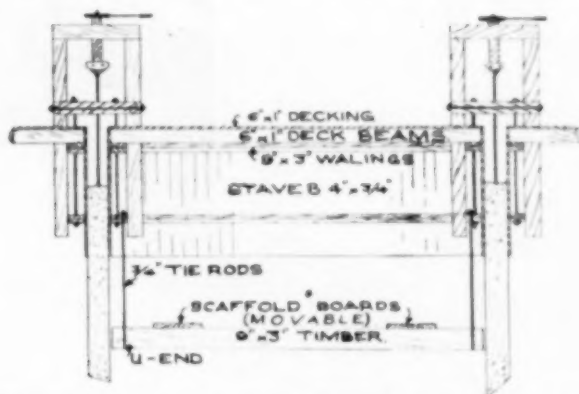


Fig. 10.—Details of Deck and Internal Hanging Scaffold.

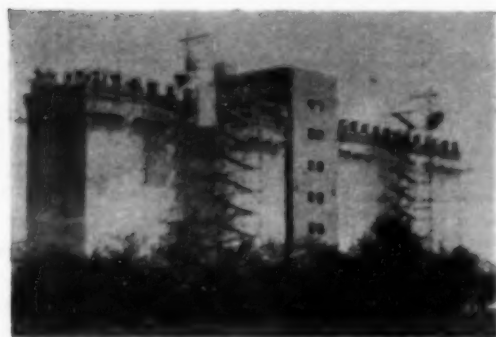


Fig. 11.—Moving-forms Used in Construction of Rectangular Bins, showing Access to Deck.

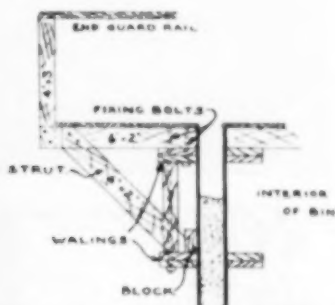


Fig. 12.—Working Platform outside the Wall.

by means of an external staircase, or by ladders, or by an internal or external ramp (as in *Fig. 11*), or by two or more of these means if the structure is large.

The deck can also be shuttered with steel plates carried on 6-in. by 2-in. timber joists at 1-ft. centres, but if the bin is circular the parts of the deck adjacent to the wall must be completed with timber. If 2-ft. square proprietary plates are used the timber joists need to be at only 2 ft. centres, thus enabling the

\* Continued from March and April.

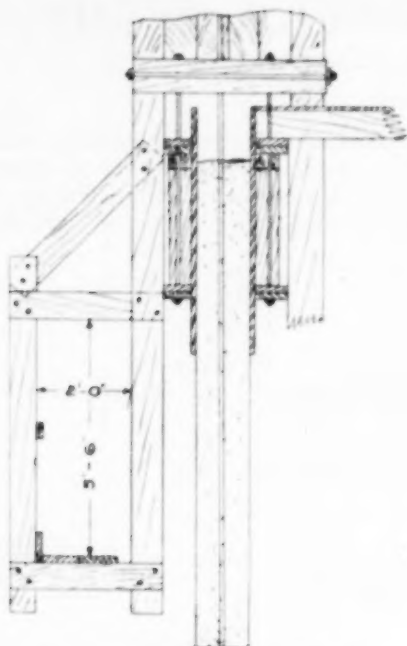


Fig. 13.—Hanging Scaffold below Forms.

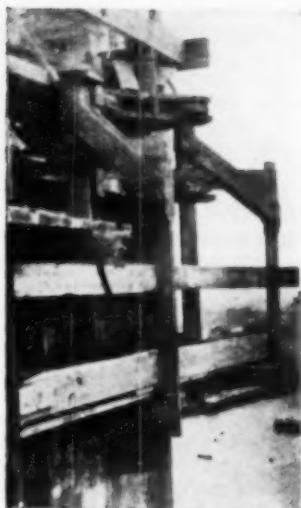


Fig. 14.—Hanging Scaffold below Forms.

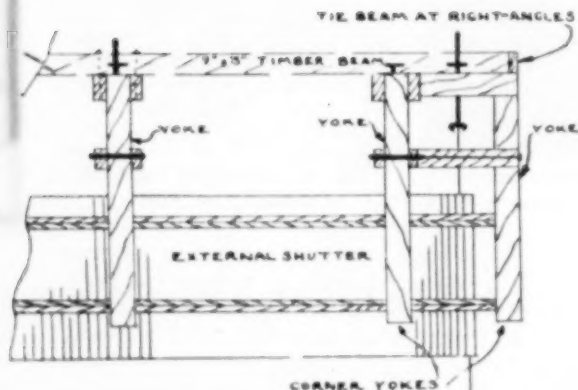
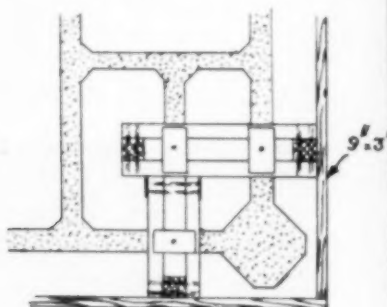
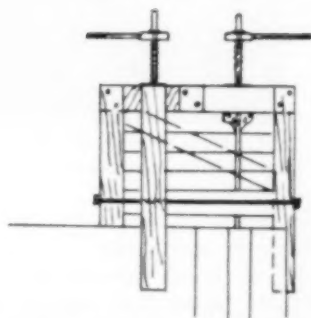


Fig. 16.—Cross-beams to Prevent "Dragging" at Corners.



PLAN



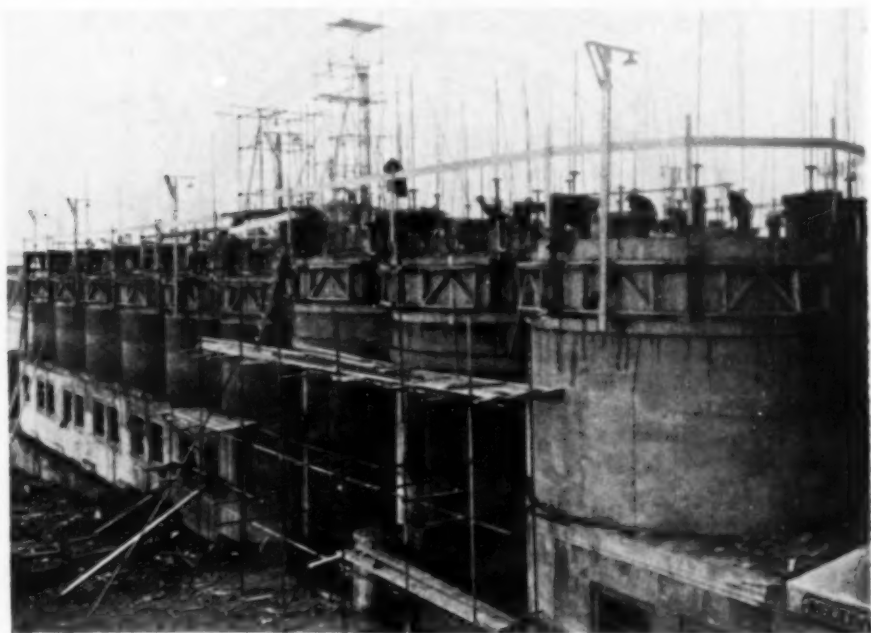
ELEVATION

Fig. 15.—Special Yokes at Corners to Prevent "Dragging".

May, 1950.

provision of a manhole which is essential for dismantling purposes and also for access to the wall forms if any constructional difficulty arises. Adequate provision must be made for the bearing of the joists on the walings.

Around the outside of a silo or similar structure it may be that, due to the small dimensions of a particular bin, the space for fixing reinforcement and placing concrete is restricted, and to ensure adequate space a cantilever balcony may be fixed on the external form to the walls. *Fig. 12* shows such a form and, provided that the balcony is rigid, the twisting effect on the outer form will not be sufficient to produce binding on the face of the outer wall.



**Fig. 17.—Moving Forms Used in Construction of Circular Bins.**

#### Scaffolds.

Access to the outside of the walls is achieved by suspending from the outside forms a hanging scaffold sufficiently wide to allow men to work on the walls immediately below the bottom of the forms. *Figs. 13* and *14* show a typical hanging scaffold; it is hung from the yokes, and comprises strong boards with scaffold-boards spanning between the cross members and with a handrail and toe-board. The provision of good means of access to the outside walls is necessary since time is limited in which the walls can be rubbed down or made good, because the hanging scaffold rises as the forms move upwards. From the scaffold the surface of the external walls can be kept under constant surveillance.

Similarly it is necessary to provide an internal scaffold, which can be carried on hook-rods suspended from the lower walings (*Fig. 10*). An internal scaffold

usually comprises timbers placed in the hooks at the bottom of the rods, and across these timbers are laid one or two scaffold boards. This scaffold also rises with the forms.

#### Preventing "Dragging" at Corners.

The forms for large square and rectangular bins tend to drag behind at the corners when the forms are moving upwards. If a bin is, say, 15 ft. square,

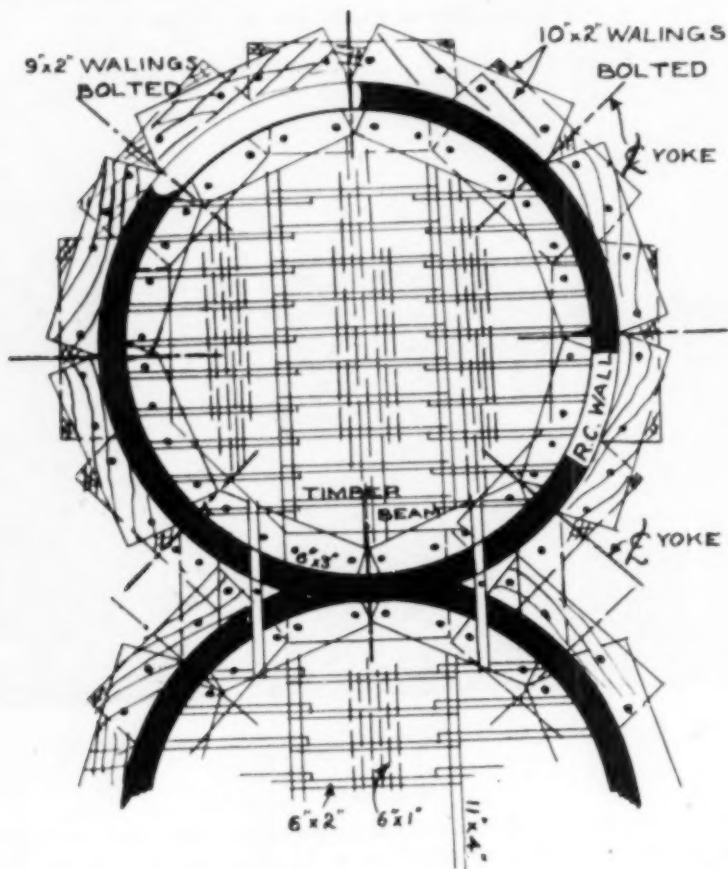


Fig. 18.—Details of Moving Form for one Row of Circular Bins.

and two yokes, at about 7 ft. 6 in. centres, are provided for each wall, the corners are nearly 4 ft. from the yokes. The friction of wet concrete on the forms tends to hold down the corners. To counteract this, it is usual to provide stiff bracing to the internal walings, or to provide special corner yokes as in Fig. 15 when the number of ordinary yokes is not sufficient to provide the necessary stiffness. Sometimes it is possible to provide cross-beams as in Fig. 16 along the top of

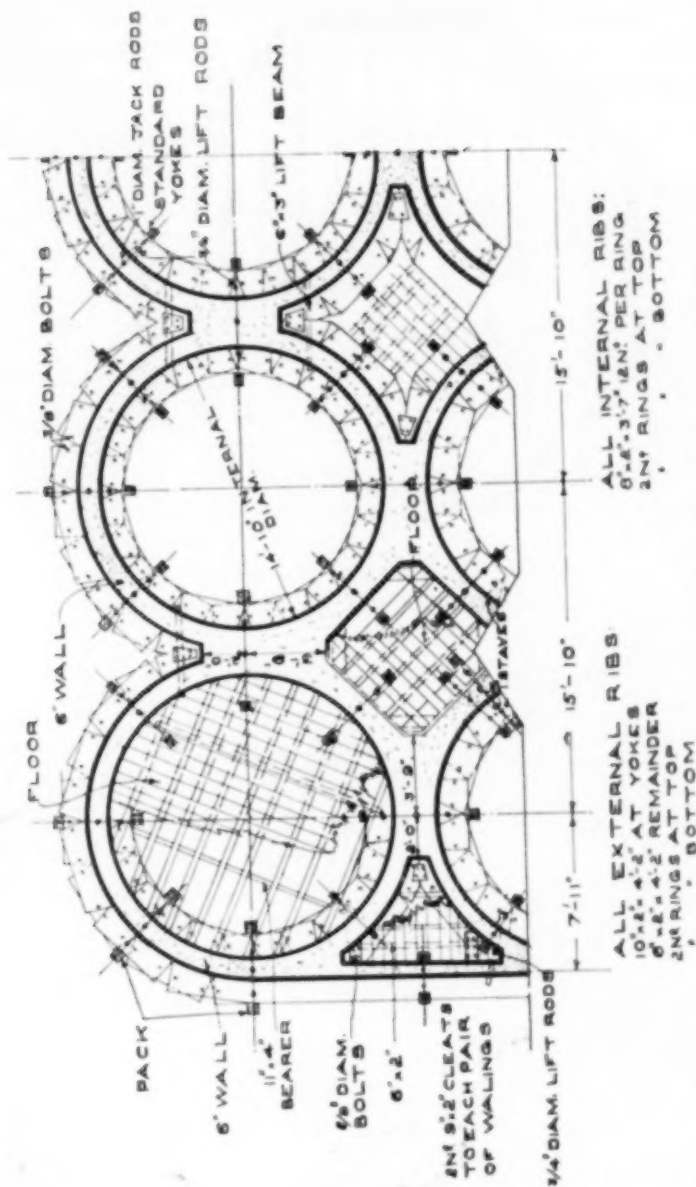


Fig. 19.—Moving Forms for Two or More Rows of Circular Bins.

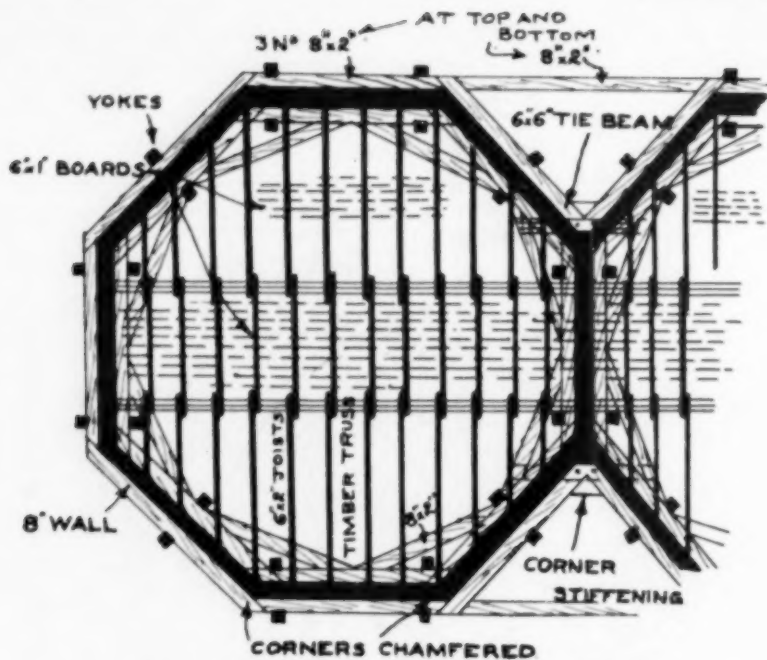


Fig. 20.—Moving Forms for Polygonal Bins.

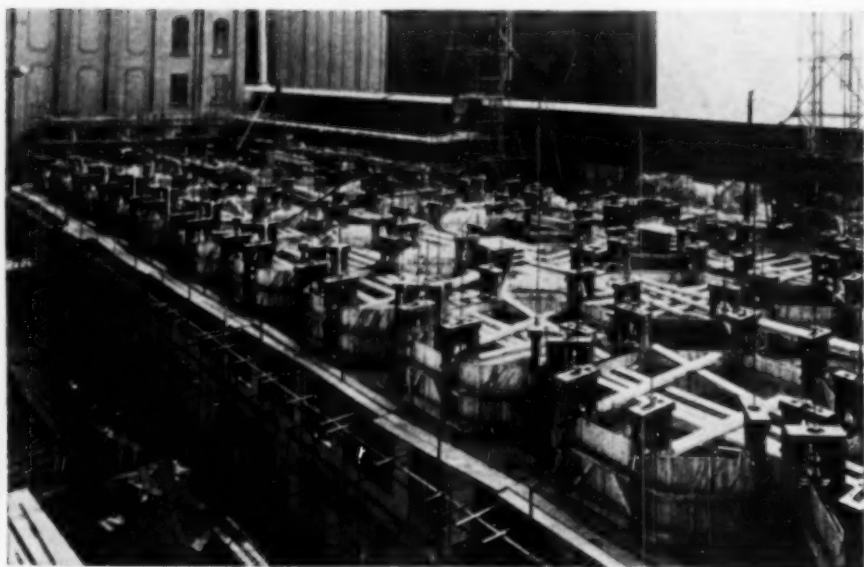


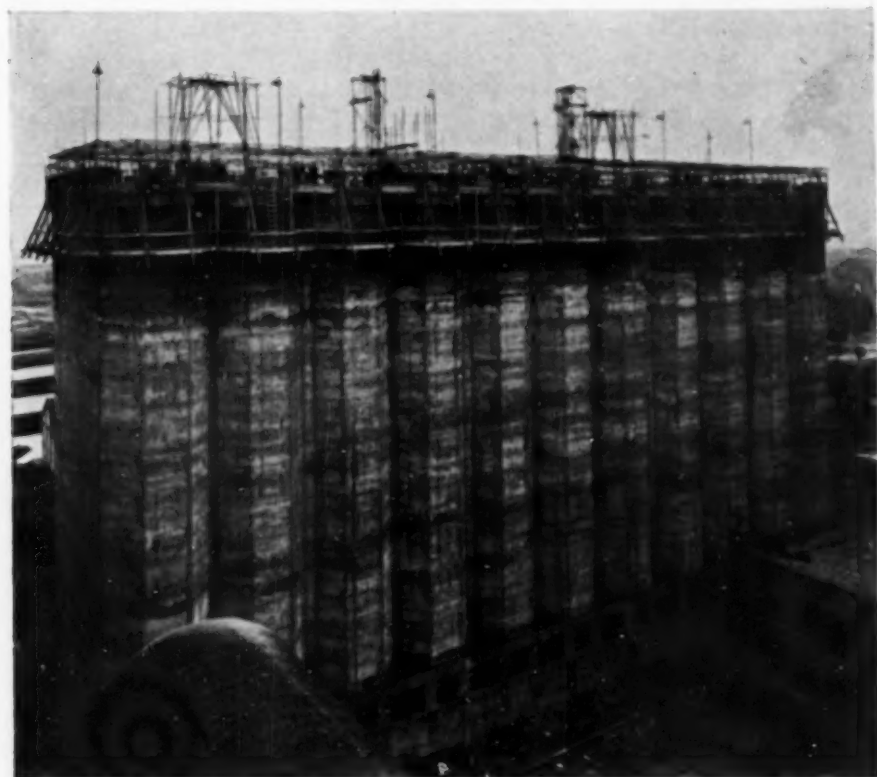
Fig. 21.—Moving-Forms on Octagonal Silos.

the first three or four yokes. The cross-beam is bolted to the top of the yokes and distributes evenly the tendency of the corner to drag.

### Forms for Circular and Polygonal Bins.

Although the foregoing notes apply mainly to rectangular and square bins, most of them apply also to circular and polygonal bins.

The forms for circular bins (*Fig. 17*) do not tend to warp as much as those for square or rectangular bins, although twisting of the forms by careless jacking



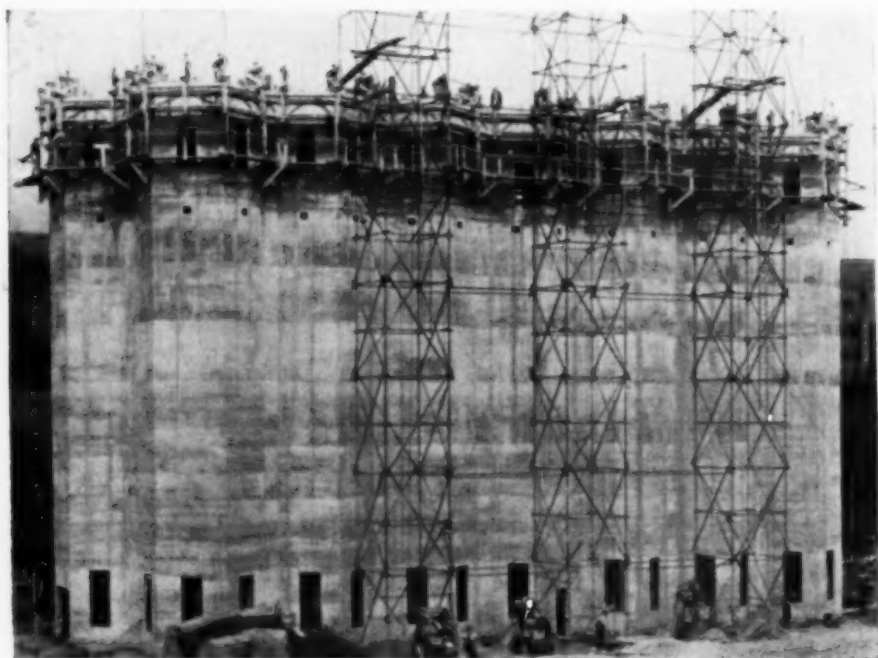
**Fig. 22.—Moving-Forms in Use on Octagonal Silos.**

can cause jamming against the walls. Details of the forms for a single row of circular bins are shown in *Fig. 18* and for two or more rows in *Fig. 19*. The walings are made of short lengths of timber, and the staves, being very narrow, are able easily to conform to the curvature without difficulty.

The forms for polygonal bins (*Fig. 20*) are more troublesome to make, but have the same strength and freedom from jamming as moving forms for circular forms. Corner beams are essential at the junctions of the bins to prevent the corners from drooping. *Figs. 21* and *22* are two views of the construction by moving forms of grain silos comprising octagonal bins. In *Fig. 21* moving-



AFTER FIVE DAYS.



COMPLETION AFTER 9½ DAYS.

Figs. 23 and 24.—Moving Forms in Construction of Hexagonal Bins.

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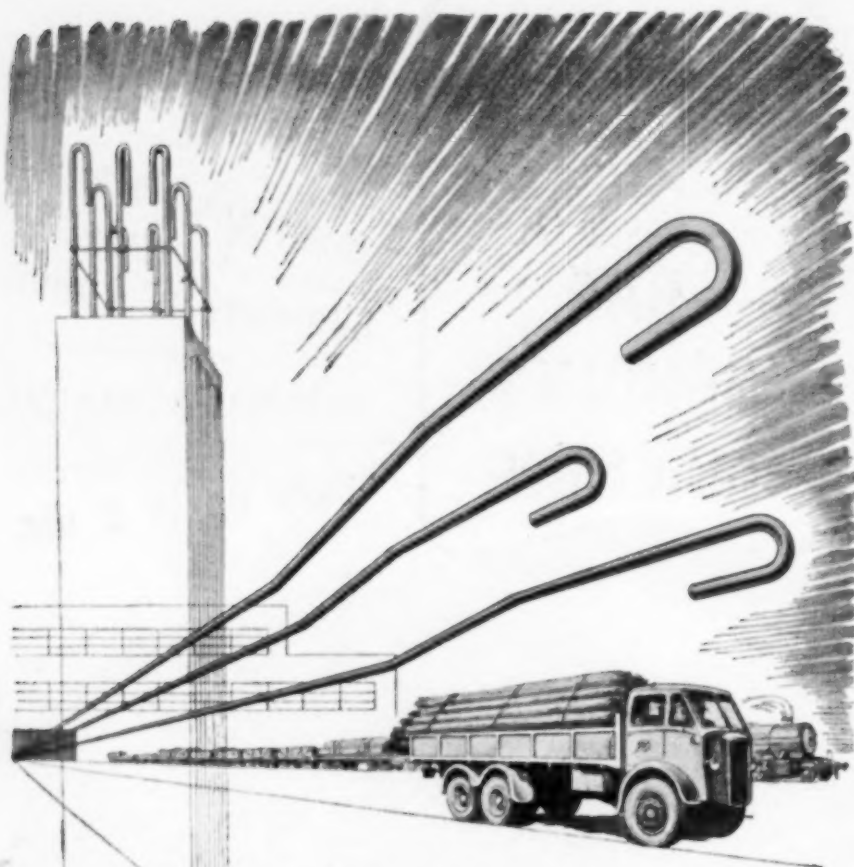
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form construction is about to commence at the top of the structure below the bins. The height of the walls constructed in this manner is about 85 ft., and the work is nearing completion in Fig. 22. Figs. 23 and 24 show a coal bunker having hexagonal bins in course of construction.

### Lateral Bracing of Forms.

It is important to tie together the tops of the forms for each bin. It is true that the yokes restrict outward movement of the whole of the forms, but the

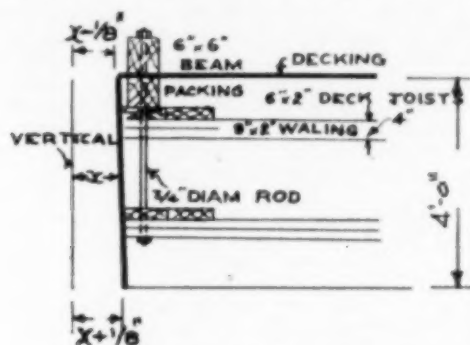


Fig. 25.—Strut at Top of Forms.

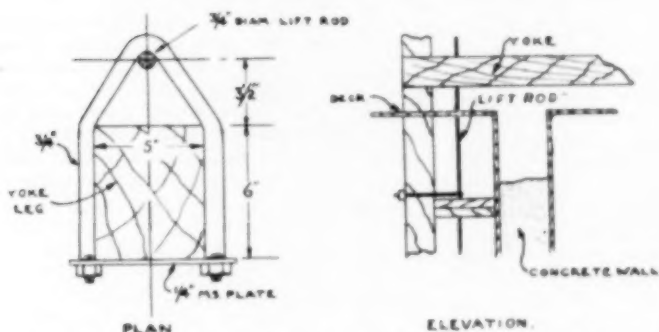


Fig. 26.—U-Bolts to Prevent the Legs of the Yokes from Spreading.

connection to the walings is only by the vertical lift-rods. Although horizontal movement is restricted by the grip of the forms on the walls, side movement can and does occur unless the tops of the forms are tied together by short beams or struts across the width of the walls; it is impossible to guarantee the width to be constant unless this is done. Fig. 25 shows a cross-section of such a beam or strut with a tie-rod extending to the bottom waling.

Fig. 26 shows another precaution necessary and consists of a U-bolt holding the legs of the yokes to prevent them from spreading. It also helps to prevent the yokes tilting due to unbalanced strain on the forms.

(To be continued.)

## Some Developments of Prestressed Concrete.

The following notes are abstracted from a series of lectures arranged by the Prestressed Concrete Development Group and delivered at the last Building Exhibition held in London.

### Stretched Wires for Prestressed Concrete Tanks.

The method used by the Preload Corporation of America to maintain the tension in high-tensile wire used in conjunction with a machine which, while propelling itself round the tank wraps the wire around the wall, is described in the following. The operation of wrapping starts at the bottom and proceeds upwards, the spacing of the wires being adjusted to suit the resistance required. Tensioning of the high-tensile wire is

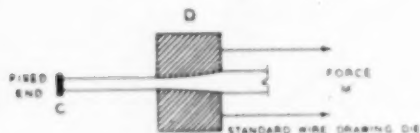


Fig. 1.

effected by extruding it through rollers or a wire-drawing die. A standard wire-drawing die is used in conjunction with the wrapping machine. If a wire is stretched its diameter is reduced. If the end of the wire (C) in Fig. 1 is fixed and a pull is exerted on the die (D) the wire will stretch and the die will move along the wire as soon as the diameter has been reduced sufficiently to pass through the hole in the die. The size of die required to produce a stress of 140,000 lb. per square inch in the wire is 0.142 in. diameter for 0.162-in. diameter wire used in America, and 0.177 in. diameter for the 0.2-in. diameter wire used in this country. Theoretically, stretching the wire and reducing its diameter sufficiently to allow it to pass through dies of the sizes stated would not produce a stress of 140,000 lb. per square inch in the wire. If the heat produced by pulling the die along the wire at a speed of 4 miles to 7 miles an hour is taken into consideration, the stress, when the wire has cooled will be that required. Since the tension is induced in the wire before it comes in contact with the wall of the tank, there is a uniform tensile

stress in the wire and no variations due to friction between the concrete wall and the wire.

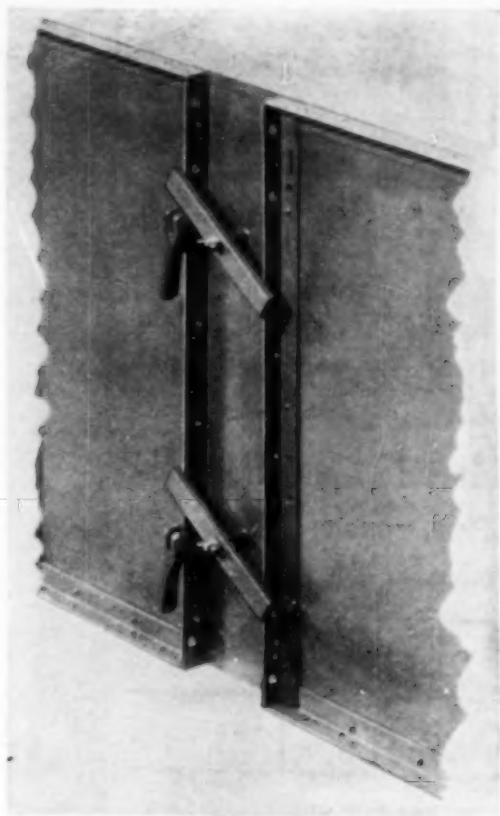
### Anchorage Cones.

The inner cone of the two cones used in the Freyssinet system had in the first type a concrete body with high-tensile steel wire wrapped around it similar to the outer cone into which it was forced. The stretched wires of the prestressing cables were thus held between two steel-faced surfaces, and precision of the surfaces and shape of the wire were necessary for good contact and effective anchorage. Sufficient precision was obtained if cold-drawn wires were used, but if heat-treated wires, which are less exact in shape, are used the anchorage by the steel-faced inner cone is not reliable. This type of cone has now been replaced by a concrete plug with grooves in its side to receive the wires of the cables, the plug being lightly reinforced with a small steel gauze and nails. The plug is driven into the outer cone with a force of 20 tons, which produces a stress greater than the crushing strength of the concrete in the plug if it were unrestrained. The concrete of the outer cone seems to flow around the wires and grips them thoroughly, and is restrained by the effect of the steel helices of the outer cone and the nails in the plug, the heads of the nails lying on the outer end of the cone. The plug anchors wires of slightly irregular shape, and it provides a better anchorage than its predecessor since the friction of steel on steel is replaced by the greater friction of steel on concrete; moreover the yield of the anchorage when it receives its load as the pull of the jack is released is smaller and does not exceed 0.05 in. to 0.10 in.

### Grouting of Cables.

In the Freyssinet system, the grout around the cables is injected under a pressure of 80 lb. to 90 lb. per square inch. Such a high pressure is possible because the aperture is small and consequently the bursting forces due to the pressure are also small. A high pressure is necessary to force the grout between the wires and completely to fill the interstices between the wires. Water is injected first, and

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then the grout is injected at one end until it comes out of the other end of the member. The injection is then reversed. The mixture generally used is 1 part by volume of cement to 1 part of sand to 1 part of water, which is a rich creamy grout. A beam 20 ft. long of which only the end 15 in. of the cables were injected with grout was tested after the grout had set and the cones had been destroyed. It failed by crushing of the concrete without any slipping of the steel. The cables of a hollow prestressed concrete pile 100 ft. long were injected with grout and the cones destroyed. The pile was then driven. The cables did not slip although the interaction of the cables and the concrete depended only on the bond of the grout.

In the Magnel system of prestressing concrete beams with cables, the cables are injected with a colloidal grout, which in Britain is a mixture of  $4\frac{1}{2}$  gallons of water to one bag of cement, that is about 2 : 1 by volumetric proportions. Test cubes made of this grout taken directly from the mixer show that there is hardly any shrinkage.

#### Members made by the Hoyer System.

The factory of Strangbetong Fabrik at Liljeholmen, Sweden, where full production commenced in 1942, has a yearly production of about 250,000 cu. ft. of precast concrete products prestressed on the Hoyer system of small wires stretched along a long bench on which several products are made at the same time. The products include beams up to 90 ft. span, 12-in. square bearing piles up to 87 ft. 6 in. long, columns, poles, roof and floor members, and numerous smaller products. The prestressed products have been very satisfactory when exposed to the action of the sea and adverse weather including extreme cold.

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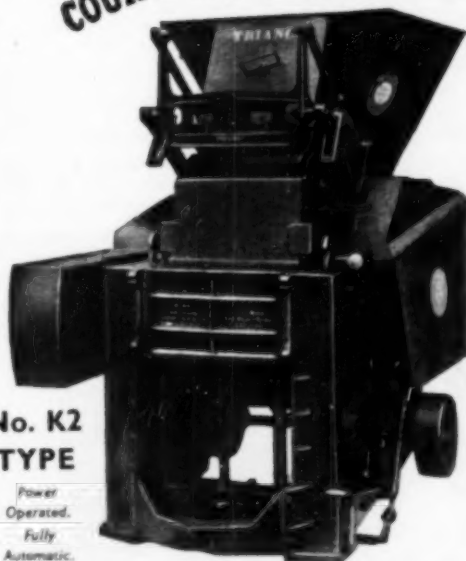
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A METHOD of transporting concrete from the mixer to the shuttering, which was designed for use in placing concrete footings and ground floors in large groups of houses and which has also been used for other purposes, is shown in the illustration. The arm on which the skip travels is 45 ft. long; it is made of a light alloy, and is in two sections each 22 ft. 6 in. long which are bolted together. The rear

required distance when the concrete is discharged through two bottom gates operated by a lever. (5) The empty skip is returned until the point of balance is reached and the boom swung back to the mixer. The cycle takes from 1½ to two minutes. The two rubber-tyred wheels on the undercarriage traverse on a metal baseplate 4 ft. 9 in. wide. The carriage is connected to the baseplate by a throw-



Fig. 1.

portion is provided with ten detachable trays, which are filled with lead to act as a counterweight. The travelling skip is operated by a double-acting hand-winch and the sequence of operations is as follows. (1) The skip travels the length of the boom, the front end of which is at the mixer, and stops between the fixed supports at the front. (2) The skip is filled directly from the mixer discharge chute and taken towards the middle of the boom until the point of balance is reached. (3) The boom is then swung round to the particular point where the concrete is required. (4) The rear adjustable leg is lowered and the skip taken forward the

over clamp and may be moved to a new position by two men in five minutes.

The illustration shows this device in use in the placing of concrete for the floors of subways at London Airport at the bottom of an open cut 70 ft. wide by 20 ft. deep. A staging of alloy scaffold tubes (Fig. 1) was erected to carry a hopper and distribution chute and to support the end of the boom. As the work progressed this staging was lifted and moved along, the mixer being moved as each 50-ft. bay was concreted. The device was designed by Messrs. George Wimpey & Co., Ltd.

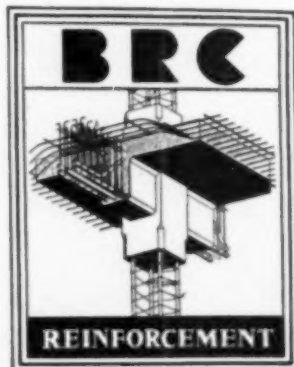
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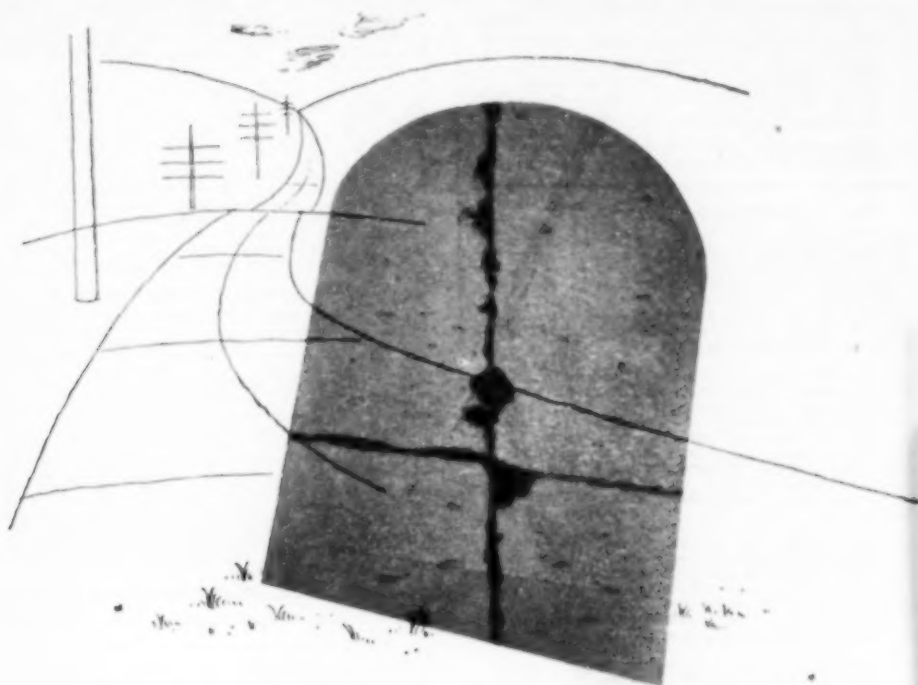
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**SITUATION VACANT.** Quantity surveyor urgently required by consulting engineers for reinforced concrete construction and general building. Duties also to include contractors' accounts. General knowledge of reinforced concrete design advantageous but not essential. Write, stating age, experience and salary required, to J. C. HUGHES & PARTNERS, 119 Marylebone Road, London, N.W.1.

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## COMMONWEALTH OF AUSTRALIA.

The Commonwealth Department of Works and Housing, which is the main constructing authority for engineering and architectural works for all Commonwealth Departments in Australia, desires to appoint qualified engineers, architects and quantity surveyors for service in Australia. Persons interested should forward applications to Mr. W. C. Alexander, Department of Works and Housing, Australia House, Strand, London, W.C.2.

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The appointment will be terminable by one month's notice on either side, and subject to the provisions of the Local Government Superannuation Acts, and the successful candidates will be required to pass a medical examination.

The County Council is not in a position to assist successful applicants with housing accommodation.

Canvassing members of the Council directly or indirectly will be a disqualification for appointment.

Applications to be made on a form to be obtained from the undersigned to whom it must be returned, accompanied by copies of three recent testimonials, not later than May 27th, 1950.

F. HAMER CROSSLEY,  
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**MISCELLANEOUS ADVERTISEMENTS**

(Continued)

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The Department of Scientific and Industrial Research is providing the organisation, and papers are being invited from research workers in many countries. The purpose of the congress is to review research in relation to architecture, building, and civil engineering, and the papers presented will deal with recent research and its influence. Visits to buildings and civil engineering works will be arranged. Further information may be had from The Building Research Congress 1951, Building Research Station, Garston, Herts.

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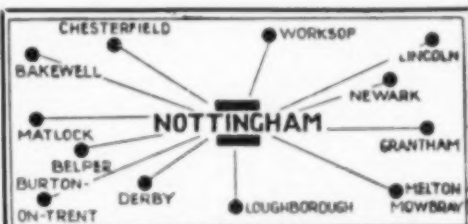
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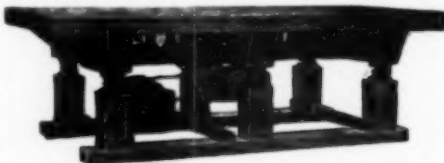
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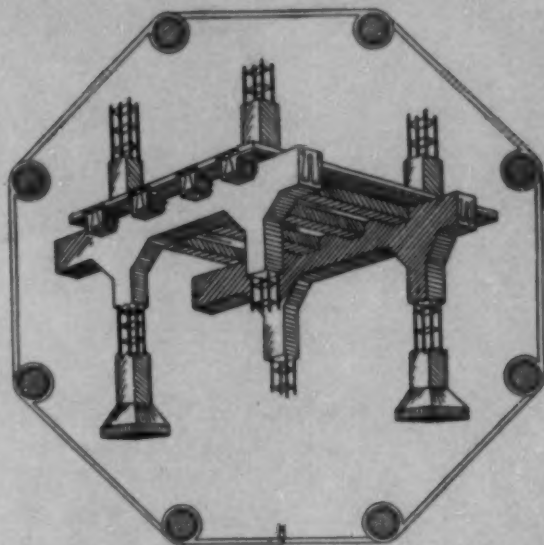
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